APPENDIX 15.C — EXAMPLES

15.C.1 EXAMPLE 1 — SMOOTH CULVERT FISHWAY

This Example illustrates the simple analysis used to identify fish migration capabilities of a culvert based on a selected flood peak. For this example, arbitrarily selecting the 10-yr flood as a reasonable and prudent engineering standard means that fish will have a 90% or better chance of migrating through the culvert each year.

All commercial pipe (corrugated or smooth) are considered as smooth for fishway design where the invert is at streambed elevation. Of importance is the shape. This Example addresses three shapes:

- round,
- arch, and
- oval.

Round Culvert

Given the following information, design a round metal culvert to meet Deaprtment flood hazard criteria, and then determine whether or not the culvert will provide for satisfactory fish movement:

Step 1, CRITERIA. Through negotiations with the responsible resource and regulatory agency(ies), the following design fish related criteria were agreed upon:

- A simple type analysis is acceptable.
- A seasonal run dictates the need for both adult and juvenile fish migration throughout the summer.
- The only species of concern is brown trout.
- Provision be made for both adult and juvenile fish to migrate (juvenile fish having a fork length of one-half the adult fish).
- The sustained speed for the adult fish is FV1 = 5 ft/s.
- The sustained speed for the juvenile fish is FV2 = 3 ft/s.
- The minimum design flow depth in the culvert is 24 in.

Most of these resource and regulatory agency recommendations were found to be consistent with Table 15-5, Table 15-6, Table 15-7 and Figure 15-3, and further negotiations regarding the design fish speeds would not be productive. The required flow depth for fish migration did seem excessive and worthy of some further negotiation in light of Table 15-8.

Step 2 HYDROLOGY. The necessary site data are obtained and the hydrology estimated:

Average stream slope = 0.004 ft/ft

Q_{25} = 175 ft^3/s
Q_{10} = 112 ft^3/s
Q_2 = 55 ft^3/s
Q_s = 2 ft^3/s – 10 ft^3/s (normal summer flow)

To satisfy Deaprtment criteria for this site, a HW/D < 1 for the 25-yr discharge of 175 ft^3/s is required, where: HW = headwater depth, ft, D = diameter of culvert, in.

For the simple method, the discharges and velocities to be used in evaluating the fish passage capabilities are:

Q_1 = (0.65)(112) = 73 ft^3/s,
Q_2 = (0.20)(112) = 22 ft^3/s,
V_1 = 5 ft/s for 65% of the 10-yr discharge and
V_2 = 3 ft/s for 20% of the 10-yr discharge.

Step 3 CULVERT SIZE. A culvert is initially sized to meet highway criteria. Based on the practices in the Culverts Chapter, it is determined that a single 84 in CMP is adequate.

Step 4 FISH PASSAGE VELOCITY CHECK. Determine if the single 84 in CMP will be adequate for fish passage. The barrel and outlet velocities for the culvert were both found to be:

For Q_1 = 73 ft^3/s, the velocity is V_1 = 5 ft/s
For Q_2 = 22 ft^3/s, the velocity is V_2 = 3 ft/s

Because V_1 is not greater than FV1 and V_2 is not greater than FV2, the design to this point is satisfactory.

Note: Use of average velocity for this check is conservative, perhaps overly conservative depending on fish species and age class. Use of a computed velocity based on a horizontal velocity distribution curve for an area nearer to the edge of water in the culvert may be more appropriate. The idea is to use a reasonable value that characterizes the flow velocity where the fish are actually swimming.

Step 5 MIGRATION FLOW DEPTH CHECK. Although the 84-in CMP meets the highway criteria, the tailwater must be maintained at an adequate depth. This can be accomplished by constructing a sill(s):

- downstream from the outlet, and
- beyond the predicted scour hole.*

**See Energy Dissipator Chapter for predicting scour hole geometry.*

The top of the sill is set to provide a depth and hence velocity through the culvert that is equal or less than FV1 and FV2; see Figure 15-6. Determining a sill elevation is discussed later. If the channel is stable below the outfall (bed rock outcrop or a stream bed heavily armored with large boulders), it may not be necessary to construct a sill if the only problem is the formation of a scour hole that would preclude migration. In Step 4, assume that it was shown, barring a headcut, a sill was not required.

However, if this culvert had been on a 1% slope (instead of 0.4%), the normal velocity for Q = 73 ft^3/s would exceed the 5 ft/s allowable and the 3 ft/s allowable for Q = 22 ft^3/s. Under these conditions, consider:

- alternative designs, or
- downstream sills.

Note: With a sill(s), it would be necessary to determine whether it is feasible to construct a downstream sill(s) of sufficient height so that the backwater through the culvert would increase depths and lower all velocities (inlet, interior, outlet) to acceptable levels.

The backwater induced at the inlet would probably be the critical point unless a hydraulic jump forms in the barrel to preclude fish passage. For this site, first try the sill(s) alternative.

First try a single sill. (From this point on, the following discussion is substituted for actual computations; sill design is addressed in more detail in Example 3). The elevation of the sill shall be established by backwater computations. The depth for Q = 73 ft^3/s (or Q = 22 ft^3/s) is determined for the crest of the sill. The weir equations provided in Section 15.4.10 can be used to estimate the flow depths over the sill. Using this depth as a starting point, the water surface profile through both the pool and culvert is estimated with backwater computations; i.e., a trial elevation of the weir crest is selected and the depth at the culvert entrance is obtained by backwater computations through the culvert; see Culverts Chapter. Again, note that it will be necessary to extend these computations through the culvert (particularly long culverts) so as to verify that:

- a hydraulic jump does not occur at the outlet, in the culvert, or at the inlet; and
- both velocity and flow depth at these points are within acceptable limits.

Where this computed depth (1) matches the required fish migration depth (2 ft in this case), (2) no hydraulic jumps are encountered, and (3) the velocities are acceptable, the sill is assumed to be set at the proper elevation.

Fish passage over the sill may also be required during periods of minimum summer flows and during design flows. This may require a notched sill. Further, the sill must be constructed with sufficient stability to:

- withstand the force of the design flood,
- be secure from lateral erosion (bypassing), and
- accommodate streambed scour due to the design flood.

Step 6 FLOOD HAZARD REVIEW. The culvert must also be reviewed for the design and review discharge used in the culvert design considering the effect of the sill(s). In some instances, the sill(s) may change the culvert's control from inlet to outlet; see Culverts Chapter. Should this occur, the culvert size may have to be increased to avoid violating the selected headwater criterion and thereby causing a flood hazard.

Alternatives

There are instances where a downstream sill(s) may not suffice. This usually occurs when a culvert is too:

- long, and/or
- steep.

Under these conditions, the inlet and/or interior depth and velocity may not be influenced by the sill. Costs associated with the sill and the difficulty encountered in assuring fish passage over the sill at both high and low flow(s) may also jeopardize selection of this fishway. Alternatives are to:

- revise the culvert size,
- revise the culvert geometry, or
- select another alternative.

Arch/Oval Culvert Shape

The analysis of fish passage through an arch or oval culvert shape is similar to that for a round culvert; however, velocities at low flow are generally somewhat higher due to less wall friction. Also, during low-flow periods, water depths within the pipe arch culvert may be insufficient for fish passage. If outlet control is possible, oversizing the culvert so that it can be countersunk ((2 ft)) below streambed elevation may be an adequate solution. The velocities and depths within the pipe arch must still be limited to the design fish(es)' swimming capabilities during the movement periods(s).

Substrate material for a countersunk culvert will have more roughness than a bare metal pipe. This will necessitate an iterative computation of a composite roughness coefficient for hydraulic calculations. The iteration may be performed by first assuming a roughness value for the bare culvert to compute an initial depth, then compute a composite roughness weighted by the proportion of wetted perimeter of metal culvert material to substrate material. Repeat the computations until the assumed water depth equals the depth computed using the composite roughness value.

15.C.2 EXAMPLE 2 — SMOOTH CULVERT FISHWAY

This Example illustrates a rigorous analysis to identify migration discharges and time periods using runoff records. This type of analysis may be needed where fish migration and peak flows coincide:

Step 1 CRITERIA. Determine criteria for the:

- movement period,
- analysis type, and
- design fish.

Migration Period. It is necessary to size a culvert to pass the design flood and provide for a fish spawning run during the months of May through June and for the month of October. On this stream, May through June are the months of maximum discharge which, unfortunately, corresponds to a spawning run. October is the month of minimum discharge and, again, it occurs during a spawning run.

Analysis Type. Negotiations with the responsible resource and regulatory agency(ies) resulted in their rejecting the findings from a simple analysis (Example 1). These agencies agreed that a more rigorous analysis based on records from near the site or similar sites would be acceptable. This was agreeable to the Deaprtment as a preliminary simple analysis resulted in large, costly culverts.

Design Fish. Negotiation with the responsible resource and regulatory agency(ies) finally resulted in their acceptance of a:

- sustained swimming speed of 4 ft/s;
- darting swimming speed of 10 ft/s (from Tables 15-6 and Table 15-6);
- minimum flow depth of approximately 1 ft (from Table 15-8);
- maximum jump height of approximately 1 ft (from Table 15-9); and
- fish migration being possible during spring discharges approximately equal to the average daily flow during the mean annual flood peak (assume $Q_2 \approx Q_{2.33}$), and an October discharge equal or greater than the “mean annual” low flow for October or a flow corresponding to that occurring during migration, whichever is less.

It was also agreed that, on the average, with this mean annual flood criteria there would be a 50% chance that half of the time the annual spring runoff event would be equal to or less than this selected migration discharge.

Step 2 SYNTHESIZE HYDROGRAPHS. Synthesize the annual and daily (24-h) hydrographs for the mean annual flood for both the high- and low-flow migration periods. The substeps are to estimate the mean annual:

- high-flow period, annual hydrograph for maximum, average and minimum discharge;
- high-flow period, 24-h hydrograph;
- low-flow period, annual hydrograph for minimum discharge; and

- low-flow period, 24-h hydrograph.

If a suitable culvert configuration can be devised where the (1) design flood meets the Department's criteria and (2) mean annual flood will meet the fish criteria noted in Step 1, then the culvert geometry should be acceptable.

Because there is no stream gage located at or near the site of interest, simple annual runoff hydrographs for just the migration periods will have to be "synthesized" from other gaged data in the same hydrologic region. Several methods could be used to synthesize an annual runoff hydrograph. In this Example, the annual runoff hydrograph was "synthesized" using a relatively simplistic approach as suggested in Section 15.5.5.2.

High-Flow Period, Annual Hydrograph (Maximum Discharge). Daily peak discharge data were located for three gages in the same hydrologic region, for a relatively wet year (worse case). Some of the more important runoff producing parameters such as mean basin slope, channel slope, soils and vegetation, and any others an Agency would want to list were compared with the same variables and parameters in the watershed of interest and found to be similar. Drainage area was omitted as nondimensionalizing. This Example will exclude this variable. The daily peak rates are shown in Tables 15.E-1 and 15.E-2 for the high-flow period (May-June). This same table also shows the nondimensionalized daily maximum annual flood hydrograph. This was obtained by dividing the maximum discharge for the day by the maximum discharge occurring during the selected runoff period for that gage. Figure 15.C-1 and Figure 15.C-2 is a plot of these data. The plots were adjusted laterally to align the peak discharge which must be equal to unity for each of the three gages. If extensive lateral shifts had been required, then the gage selection should be re-evaluated.

Next, visually plot a dimensionless "best fit" annual runoff hydrograph for the periods of interest as shown in Figure 15.C-1 and Figure 15.C-2 based on the plots of the dimensionless hydrographs for the three selected gages. This selected design hydrograph is also plotted on this Figure. Note: Do not just routinely average all the ordinates of the selected gages for a particular day, but use prudent judgment in devising a simple but reasonable representation of an annual runoff hydrograph for the high-flow migration period. The key is to try and portray a general trend of daily runoff. A best-fit plot where the daily discharges, and thus flow velocities, tend to be on the high side would, later in this Example, result in an estimate of fewer days being available for migration than actually occurs. Conversely, a low estimate of the daily discharges will infer fewer days where flow depths may be sufficient for migration than actually occurs.

High-Flow Period, 24-H Hydrograph. The hourly peak data for a typical 24-h period during the May-June time frame was obtained for these same three gages. Similar to the foregoing annual peak discharge, the 24-h hydrograph is also nondimensionalized as shown in Table 15.C-3 and Table 15.C-4. The data in Table 15.C-3 and Table 15.C-4 was plotted on Figure 15.C-3 and Figure 15.C-4. From the Figure, it was estimated that the minimum daily peak would be approximately 0.26% of the maximum daily peak. Similarly, the average daily peak would be approximately 0.45% of the maximum daily peak.

High-Flow Period, Annual Hydrograph (Average and Minimum Discharge). To obtain the mean annual (Q_2) hydrograph for the May-June high-flow migration period and the average daily peak, multiply the nondimensional ordinates on Figure 15.C-1 by the mean annual discharge (estimated to be Q_2 = 455 ft^3/s) and the average daily peak factor of 0.45 determined from Figure 15.C-3. Plotting these now dimensionalized values for the average daily mean annual peak discharge for the May-June period (high-flow period) results in Figure 15.C-5. Similarly, the annual hydrograph for the October minimum discharge can be obtained using Figure 15.C-4.

Low-Flow Period, Annual Hydrograph (Minimum Discharge). A reasonable low-flow hydrograph for the October migration period is determined in the same manner. Only in this instance the maximum daily peak discharge, although from the same three gages, was (1) selected for October from relatively dry years (worse case), and (2) the maximum annual discharge for the dry flood period was divided into the daily discharges (rather than the mean annual flood as with the high-flow period) to nondimensionalize the daily annual flow hydrograph for October. Also, rather than use the mean annual flood (Q_2), which is based on the maximum daily spring discharge for the year, a “mean annual” flow for the minimum daily discharge in October is determined as a function of the maximum daily discharge during the October low-flow periods shown in Table 15.C-2. These minimum daily annual discharges are plotted on Figure 15.C-2. The synthesized, nondimensional 24-h hydrograph shown on Figure 15.C-4 was plotted from typical, nondimensionalized 24-h hydrograph data for the three gages; see Table 15.C-2 and Figure 15.C-4. Again, as with high flows, the hourly discharge for that day was divided into the maximum daily discharge for the same day. As with the high-flow procedure, the minimum daily discharge factor of 0.48 is determined from Figure 15.C-4. The average, low-flow annual hydrograph could be determined in a similar manner. As noted in Step 1, the ability of the fish to migrate in the natural channel during low-flow periods may dictate that a greater low-flow discharge be used for design.

The data in the first column of Table 15.C-5 is the maximum daily discharge for each gage during the low-flow period (October) for all the years of record. A Log Pearson III analysis of each gage’s data resulted in an estimate of the maximum mean annual flow rate for that gage as shown in Table 15.C-6. Also shown in Table 15.C-6 is the drainage area for these three gages. By interpolation/extrapolation using the drainage area regression coefficients for the mean annual flood, 12.3 ft^3/s is estimated below as the maximum daily “mean annual” *discharge* (Q_2) for the site in question during the October low-flow period only (Note: This “mean annual” discharge for October will usually be significantly lower than the maximum or true mean annual *flow*, Q_a, that would be estimated for the entire year). The computations are as follows:

- First, from the foregoing Log Pearson III analysis, the mean annual discharge for the low-flow period only (October) for the selected gages was determined. These findings are shown in Table 15.C-6.

- Next, from the USGS flood studies, the mean annual runoff predicting equation (for the entire year) for the site is determined to be (($Q_2 = KA^X$, where: $X = 0.6A^{-0.5}$)).

If A_1 is the drainage area at the site of interest (120 mi^2), A_2 is the drainage area at the gage, and Q_2 is the gage discharge (determined previously from the Log Pearson III analysis of the gage data; see Table 15.C-6), then by proportioning the discharge at the site of interest using findings from the Utah USGS flood studies, Q_2 site would be for gage number 1:

$$Q_{2site} = [K(A_1)^{X1}]/[K(A_2)^{X2}]Q_{2gage}$$

TABLE 15.C-1 — Daily and Non-Dimensionalized Annual Runoff Data
(May-June High-Flow Period)

Day	Gage No. 1		Gage No. 2		Gage No. 3	
	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim
May 1	68	0.15	73	0.12	72	0.11
2	83	0.19	86	0.14	84	0.13
3	101	0.23	100	0.17	97	0.15
4	124	0.28	125	0.21	121	0.19
5	118	0.26	124	0.21	122	0.19
6	102	0.23	111	0.19	112	0.18
7	94	0.21	102	0.17	105	0.17
8	87	0.19	97	0.16	100	0.16
9	106	0.24	114	0.19	115	0.18
10	96	0.21	110	0.18	109	0.17
11	104	0.23	118	0.20	119	0.19
12	106	0.24	133	0.22	134	0.21
13	106	0.24	144	0.24	137	0.22
14	142	0.32	410	0.68	414	0.65
15	185	0.41	417	0.70	439	0.69
16	238	0.53	417	0.70	435	0.69
17	277	0.62	406	0.68	424	0.67
18	374	0.83	465	0.78	488	0.77
19	440	0.98	543	0.91	571	0.90
20	448	1.00	599	1.00	633	1.00
21	379	0.85	491	0.82	525	0.83
22	344	0.77	430	0.72	454	0.72
23	329	0.73	390	0.65	410	0.65
24	332	0.74	385	0.64	406	0.64
25	304	0.68	365	0.61	382	0.60
26	263	0.59	318	0.53	340	0.54
27	239	0.53	274	0.46	293	0.46
28	241	0.54	262	0.44	282	0.45
29	225	0.50	249	0.42	262	0.41
30	206	0.46	226	0.38	239	0.38
31	172	0.38	200	0.33	208	0.33

TABLE 15.C-1 (continued)

Day	Gage No. 1		Gage No. 2		Gage No. 3	
	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim
Jun 1	157	0.35	181	0.30	186	0.29
2	159	0.36	188	0.31	199	0.31
3	132	0.29	171	0.29	176	0.28
4	117	0.26	142	0.24	146	0.23
5	108	0.24	128	0.21	131	0.21
6	105	0.23	124	0.21	130	0.21
7	94	0.21	110	0.18	115	0.18
8	87	0.19	102	0.17	106	0.17
9	79	0.18	95	0.16	100	0.16
10	71	0.16	88	0.15	92	0.15
11	65	0.15	81	0.14	86	0.14
12	59	0.13	73	0.12	77	0.12
13	57	0.13	67	0.11	72	0.11
14	54	0.12	70	0.12	76	0.12
15	79	0.18	100	0.17	104	0.16
16	66	0.15	98	0.16	108	0.17
17	61	0.14	77	0.13	84	0.13
18	65	0.15	90	0.15	97	0.15
19	52	0.12	73	0.12	80	0.13
20	46	0.10	64	0.11	67	0.11
21	43	0.10	58	0.10	60	0.09
22	47	0.10	58	0.10	60	0.09
23	43	0.10	54	0.09	58	0.09
24	40	0.09	60	0.10	65	0.10
25	43	0.10	55	0.09	59	0.09
26	42	0.09	65	0.10	71	0.11
27	30	0.07	50	0.08	52	0.08
28	27	0.06	43	0.07	43	0.07
29	28	0.06	34	0.06	33	0.05
30	—	—	35	0.06	31	0.05

TABLE 15.C-2 — Daily and Non-Dimensionalized Annual Runoff Data
(October Low-Flow Period)

Day	Gage No. 1		Gage No. 2		Gage No. 3	
	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim
Oct 1	2.5	0.16	4.4	0.20	3.6	0.36
2	2.5	0.16	4.4	0.20	3.7	0.37
3	2.9	0.19	4.5	0.21	3.7	0.37
4	2.9	0.19	5.4	0.25	4.0	0.40
5	3.0	0.20	6.5	0.30	4.0	0.40
6	3.0	0.20	7.7	0.35	4.2	0.42
7	3.2	0.21	8.6	0.39	4.2	0.42
8	3.2	0.21	4.1	0.41	4.2	0.42
9	3.2	0.21	10.0	0.45	4.3	0.43
10	3.7	0.24	10.6	0.48	4.4	0.44
11	4.0	0.26	11.4	0.52	4.4	0.44
12	4.3	0.28	13.2	0.60	4.8	0.48
13	4.8	0.32	13.7	0.62	4.8	0.48
14	5.1	0.34	14.1	0.64	4.8	0.48
15	5.7	0.38	15.2	0.69	5.1	0.51
16	6.3	0.41	15.9	0.72	5.1	0.51
17	7.0	0.46	16.4	0.75	5.4	0.54
18	7.5	0.49	17.1	0.78	5.7	0.57
19	8.2	0.54	17.6	0.80	6.0	0.60
20	9.1	0.60	18.0	0.82	6.3	0.63
21	9.5	0.63	18.0	0.85	6.7	0.67
22	10.0	0.66	19.2	0.87	7.2	0.72
23	10.4	0.68	18.1	0.82	7.7	0.77
24	10.6	0.70	19.0	0.86	8.1	0.81
25	11.1	0.73	19.6	0.89	8.4	0.84
26	11.5	0.76	20.2	0.92	8.7	0.87
27	12.0	0.79	21.0	0.95	9.0	0.90
28	13.0	0.86	21.5	0.98	9.3	0.93
29	13.5	0.89	22.0	1.00	10.0	1.00
30	14.2	0.93	21.5	0.98	9.6	0.96
31	15.0	1.00	20.4	0.93	9.2	0.92

TABLE 15.C-3 — Hourly Runoff Data
(High-Flow Period)

Day	Gage No. 1		Gage No. 2		Gage No. 3	
	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim
0100	150	0.25	130	0.27	160	0.25
0200	190	0.31	160	0.33	180	0.28
0300	200	0.33	210	0.43	190	0.30
0400	250	0.42	270	0.55	210	0.33
0500	260	0.47	290	0.59	250	0.39
0600	280	0.48	300	0.61	280	0.44
0700	300	0.51	320	0.66	310	0.49
0800	330	0.55	330	0.68	330	0.52
0900	390	0.65	350	0.72	360	0.57
1000	410	0.68	380	0.78	400	0.63
1100	480	0.80	400	0.82	500	0.79
NOON	510	0.85	480	0.98	580	0.92
1300	599	1.00	488	1.00	633	1.00
1400	550	0.92	450	0.92	560	0.88
1500	430	0.72	410	0.84	440	0.70
1600	320	0.53	310	0.63	330	0.52
1700	240	0.40	220	0.45	300	0.47
1800	200	0.33	210	0.43	280	0.44
1900	195	0.30	200	0.41	260	0.41
2000	190	0.32	190	0.39	230	0.36
2100	188	0.31	180	0.37	210	0.33
2200	185	0.30	175	0.36	190	0.30
2300	180	0.30	170	0.35	180	0.28
MIDNIGHT	175	0.29	165	0.34	170	0.27

TABLE 15.C-4 — Hourly Runoff Data
(Low-Flow Period)

Day	Gage No. 1		Gage No. 2		Gage No. 3	
	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim
0100	8.1	0.54	10.4	0.47	4.8	0.48
0200	8.2	0.55	10.5	0.48	5.0	0.50
0300	8.5	0.57	11.2	0.51	5.1	0.51
0400	9.0	0.60	11.6	0.53	5.2	0.52
0500	9.3	0.62	12.0	0.55	5.4	0.54
0600	9.5	0.63	13.1	0.60	5.5	0.55
0700	10.0	0.67	14.3	0.65	6.0	0.60
0800	12.0	0.80	16.0	0.73	6.5	0.65
0900	14.0	0.93	17.6	0.80	7.2	0.72
1000	15.0	1.00	19.8	0.90	8.8	0.88
1100	14.6	0.97	21.0	0.95	9.5	0.95
NOON	14.1	0.94	22.0	1.00	10.0	1.00
1300	13.7	0.91	21.0	0.95	9.2	0.92
1400	13.2	0.88	19.9	0.90	8.6	0.86
1500	12.6	0.84	18.2	0.83	7.9	0.79
1600	11.9	0.79	17.9	0.81	7.3	0.73
1700	11.4	0.76	16.8	0.76	6.9	0.69
1800	10.5	0.70	15.7	0.71	6.2	0.62
1900	10.1	0.67	14.0	0.64	5.7	0.57
2000	9.5	0.63	13.2	0.60	5.2	0.52
2100	9.0	0.60	12.3	0.56	4.9	0.49
2200	8.3	0.55	12.0	0.55	4.6	0.46
2300	8.2	0.54	11.6	0.53	4.4	0.44
MIDNIGHT	8.0	0.53	11.5	0.52	4.2	0.42

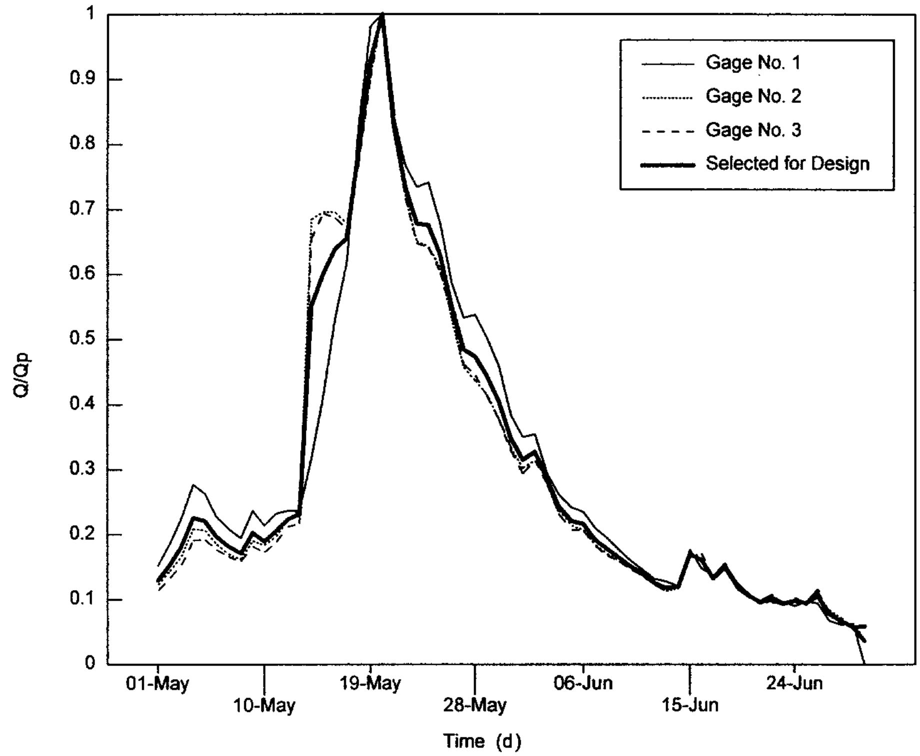

FIGURE 15.C-1 — Dimensionless Annual Runoff Hydrograph
(May - June High-Flow Period)

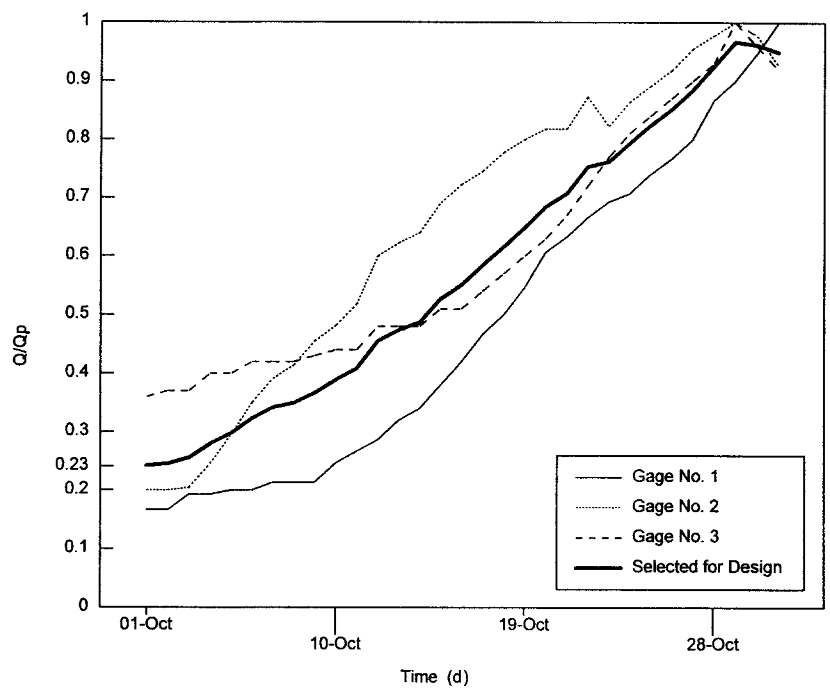

FIGURE 15.C-2 — Dimensionless Annual Runoff Hydrograph
(October Low-Flow Period)

TABLE 15.C-5 — Maximum Daily and Dimensionalized Discharges
(October Low-Flow Period)

Day	Gage No. 1		Gage No. 2		Gage No. 3	
	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim	Actual ft^3/s	Nondim
1960	8.7		12.9		12.1	
1961	9.5		18.6		11.5	
1962	14.2		17.8		10.6	
1963	15.6		19.8		9.9	
1964	12.1		13.4		12.0	
1965	13.4		14.6		12.9	
1966	10.9		11.5		8.6	
1967	18.0		17.3		7.5	
1968	17.2		14.2		6.9	
1969	8.5		9.6		5.5	
1970	15.2		10.8		7.8	
1971	14.3		14.3		9.5	
1972	11.0		13.5		10.1	
1973	8.7		13.9		11.3	
1974	9.8		16.1		9.3	
1975	10.0		11.9		7.8	
1976	16.0		15.0		11.7	
1977	12.1		14.2		10.2	
1978	13.2		20.5		9.8	
1979	16.0		19.8		8.9	
1980	14.7		18.6		10.2	
1981	14.0		21.0		9.8	
1982	15.0		22.0		10.0	

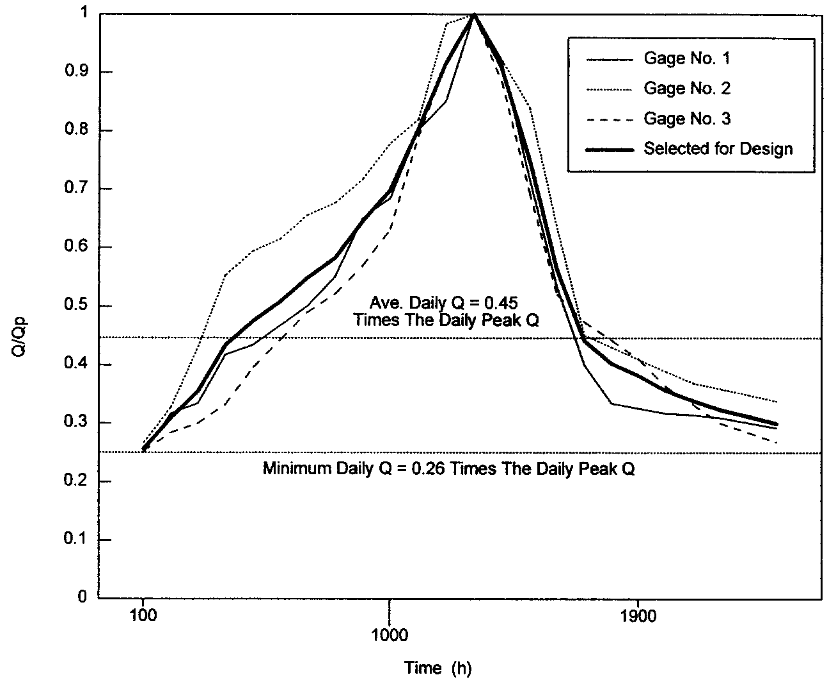

FIGURE 15.C-3 — Synthesized 24-h Hydrograph for Maximum Runoff
(May - June High-Water Flow)

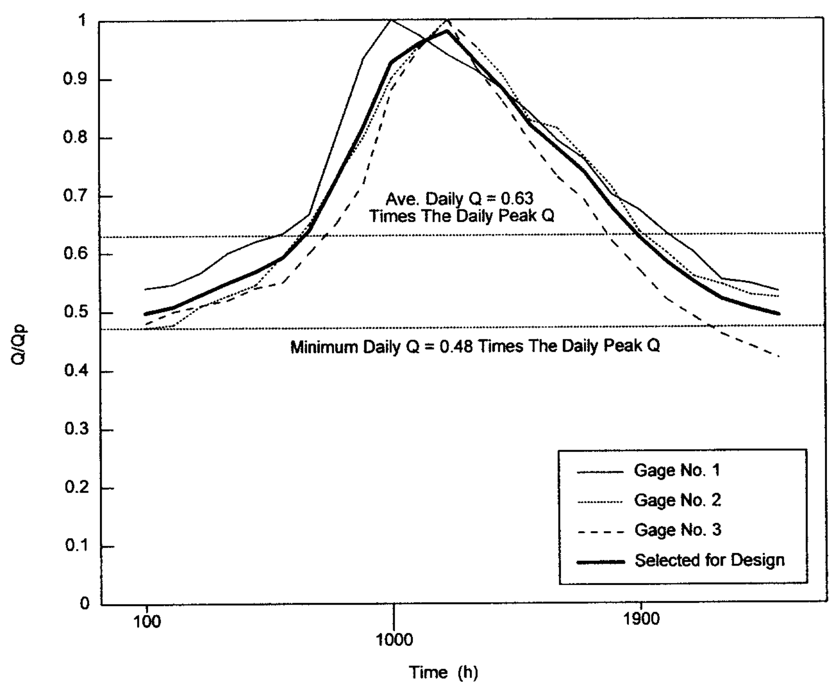

FIGURE 15.C-4 — Synthesized 24-h Hydrograph for Minimum Runoff
(October Low-Flow Period)

TABLE 15.C-6 — Mean Annual Discharge for Selected Gages
(October Low-Flow Period)

Gage No.	Mean Annual Discharge, Q_2 (ft^3/s)	Drainage Area (mi^2)
1	8.9	63
2	14.1	138
3	11.6	120
		Total 321

where: $X1 = 0.6(A_1)^{-0.05}$
$X2 = 0.6(A_2)^{-0.05}$

$Q_{2site} = [(120)^{X1}/(63)^{X2}]8.9$

where: $X1 = 0.6(120)^{-0.05} = 0.47$
$X2 = 0.6(63)^{-0.05} = 0.49$

$Q_{2site} = 11.1\ ft^3/s$

Similarly, for Gage Number 2, $Q_2 = 13.4\ ft^3/s$ and for Gage Number 3, $Q = 12.2\ ft^3/s$.

- By arbitrarily selecting a weighted average by the drainage area process to determine a reasonable Q_{2site}, we get:

Q_{2site} = 8.9(63/306) + 14.1(138/306) + 11.6(105/306)
= say approximately 12.2 ft^3/s

Recall that, in negotiations with the responsible resource and regulatory agency(ies), it was agreed to be conservative and use the minimum daily discharge in the analysis for the low-flow period. This discharge is obtained by multiplying the minimum flow factor from Figure 15.C-4 times the maximum daily discharge computed above or (0.48)(12.2) = 5.9 ft^3/s is the expected minimum daily discharge for the October low-flow period.

Also recall that, similar to the high-flow analysis, a nondimensional annual hydrograph for the October period was computed for the three gages by dividing the daily October discharges by the maximum low-flow discharge for October, and a best-fit line was plotted through the plots of the three gages as shown on Figure 15.C-2. Unlike the high-flow analysis, to obtain a synthesized annual hydrograph for October, it is necessary that the nondimensional ordinate of Figure 15.C-2 be multiplied by a factor, F, obtained by dividing the foregoing minimum daily flow of 5.9 ft^3/s by the minimum ordinate from Figure 15.C-2, or:

F = 5.9/0.23 = 25.7

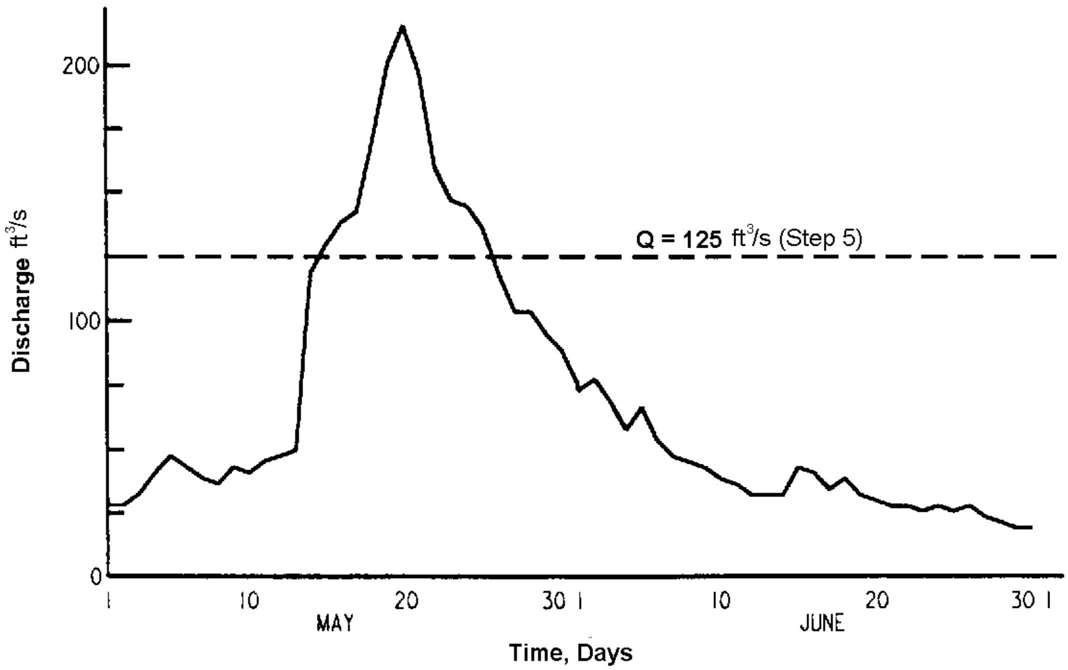

FIGURE 15.C-5 — Synthesized Mean Annual Flood Hydrograph
(May-June High-Flow Period)

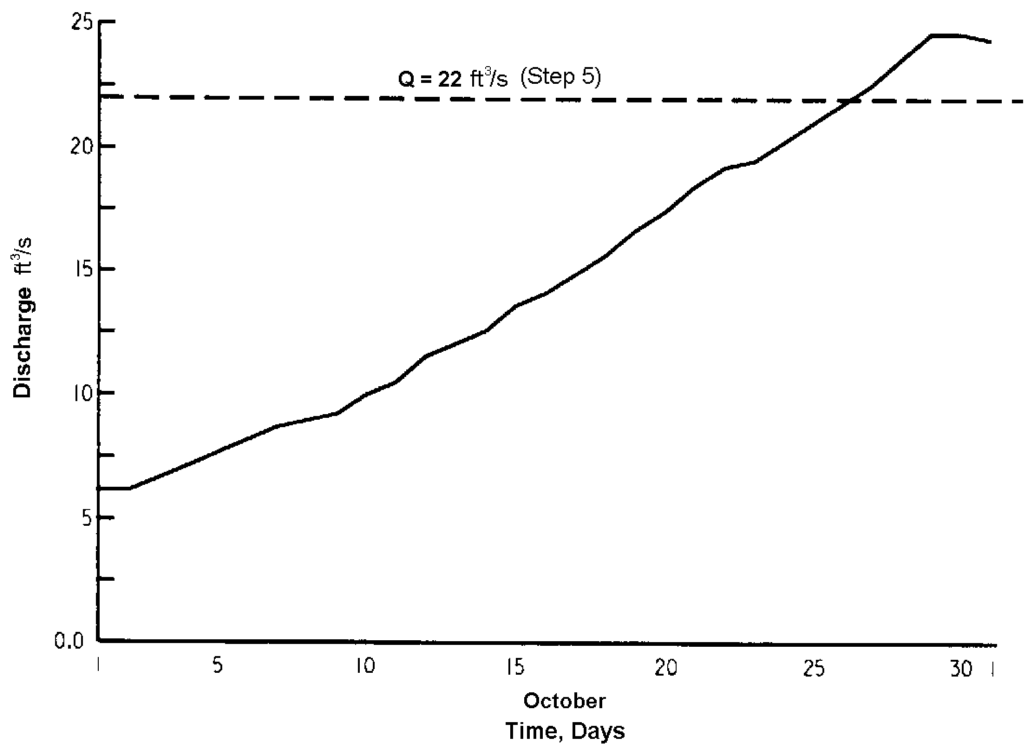

FIGURE 15.C-6 — Synthesized Mean Annual Runoff Hydrograph
(October Low-Flow Period)

The nondimensional ordinates of Figure 15.C-2 are then multiplied by this factor, F, to obtain the mean annual minimum daily hydrograph for October (low-flow period) as shown in Figure 15.C-6. Using this low-flow annual hydrograph should provide the most conservative estimate of stream flow for fish migration conditions in October. Having used the minimum "mean annual" discharge for October (not mean annual flow, Q_a), we can now say that, on the average, there would be approximately a 50% chance each year that the low flows in October would be this low or higher.

Step 3 CHANNEL HYDRAULICS. The channel hydraulics through the site's reach were estimated using a water surface profile analysis; see WSPRO in the Bridge Chapter. The cross section at the culvert site is shown in Figure 15.C-7, and the stage discharge data and stage-velocity data for the main channel are shown in Table 15.C-7. The cross section came from the site survey and the data of Table 15.C-7 were obtained with a water surface profile analysis computer program for the reach through the site. Note, the cross section reflects the armored dikes necessary to prevent the culvert's design flood from bypassing the sill where it joins the banks (see Step 8 and Example 3).

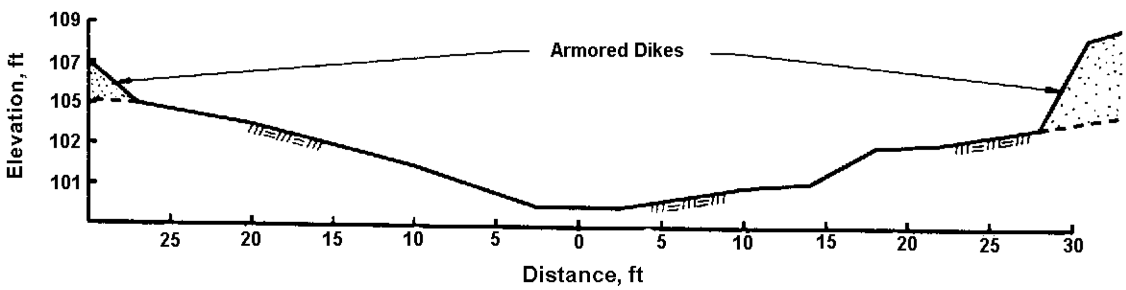

FIGURE 15.C-7 — Channel Cross Section at Site

The data in Table 15.C-7 for stage versus main channel velocity is plotted in Figure 15.C-8. Similarly, the stage-discharge curve is plotted in Figure 15.C-9.

Step 4 MIGRATION STAGES. The migration stages are estimated from Figure 15.C-8. It is essential that fish migrate during the high-flow period until a stage of 101 ft is reached. Above this stage, most of the adult design fish will probably seek cover until the velocity decreases because the velocity will exceed the selected 40 ft/s criteria.

From Figure 15.C-8, the velocity corresponding to the required minimum depth for the selected fish migration depth of 1 ft (stage = 101 ft) is approximately 2.6 ft/s. Because this velocity is less than the selected swimming speed criteria for the design fish of 4.0 ft/s, velocity will not be a problem during the October low-flow period.

Step 5 MIGRATION DISCHARGES. Next, estimate the migration design discharges. Using the stage from Figure 15.C-8 corresponding to the 4.0 ft/s criteria (102 ft), Figure 15.C-9 can be used to estimate the discharge threshold corresponding to the maximum stage for migration during the high-flood period (May-June). This discharge is found to be 125 ft^3/s and is plotted on Figure 15.C-5.

TABLE 15.C-7 — Hydraulic Properties at Site

CHANNEL SLOPE = 0.00500 ft/ft
STAGE DISCHARGE INPUT VERIFICATION

Elevation (ft)	Depth (ft)	Discharge (ft^3/s)	Velocity (ft/s)	Max. Velocity (ft/s)
99.99	0.00	0.00	0.00	0.00
100.00	0.01	0.00	0.10	0.10
100.30	0.31	2.75	1.36	1.50
100.60	0.61	10.01	1.98	2.25
101.00	1.01	27.92	2.62	3.07
101.30	1.31	48.52	3.02	3.60
101.50	1.51	66.24	3.27	3.93
101.80	1.81	103.96	3.82	4.63
102.00	2.01	133.45	4.13	5.02
102.30	2.31	183.09	4.52	5.52
102.60	2.61	241.19	4.87	5.98
102.90	2.91	317.31	5.35	6.60
103.20	3.21	403.48	5.77	7.15
103.50	3.51	500.13	6.14	7.67
103.80	3.81	604.60	6.43	8.10
104.00	4.01	681.92	6.60	8.39
104.30	4.31	814.29	6.90	8.87
104.60	4.61	959.45	7.19	9.31
105.00	5.01	1172.94	7.56	9.87
105.30	5.31	1351.41	7.86	10.30
105.60	5.61	1541.87	8.16	10.70
105.90	5.91	1743.94	8.46	11.09
106.20	6.21	1957.33	8.76	11.46
106.50	6.51	2181.76	9.05	11.83

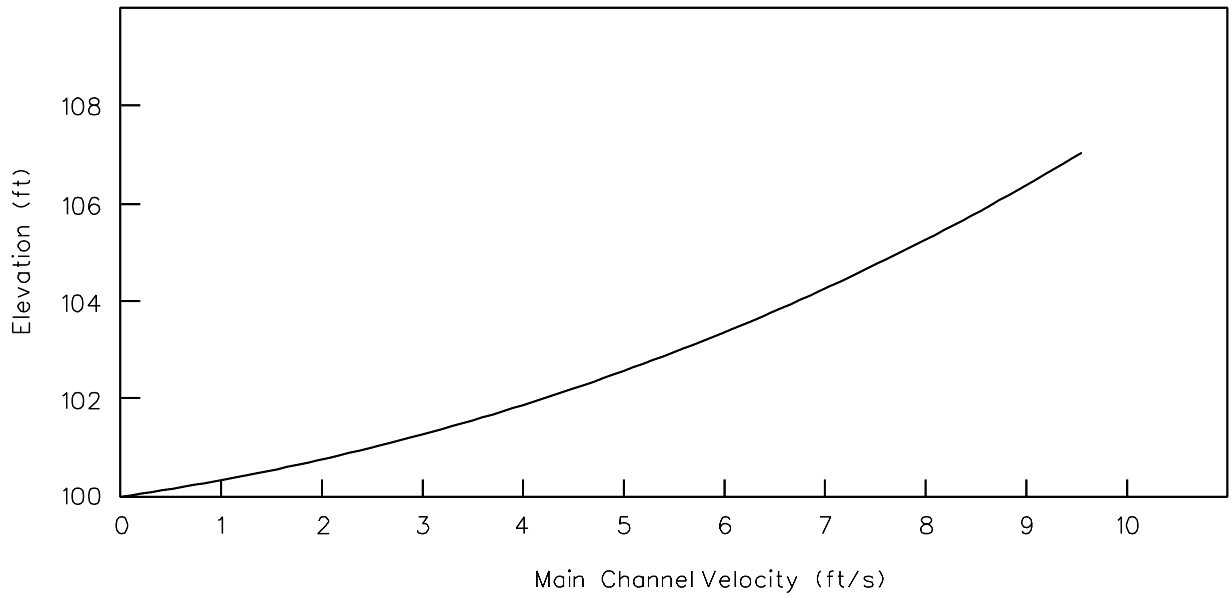

FIGURE 15.C-8 — Stage vs. Main Channel Velocity

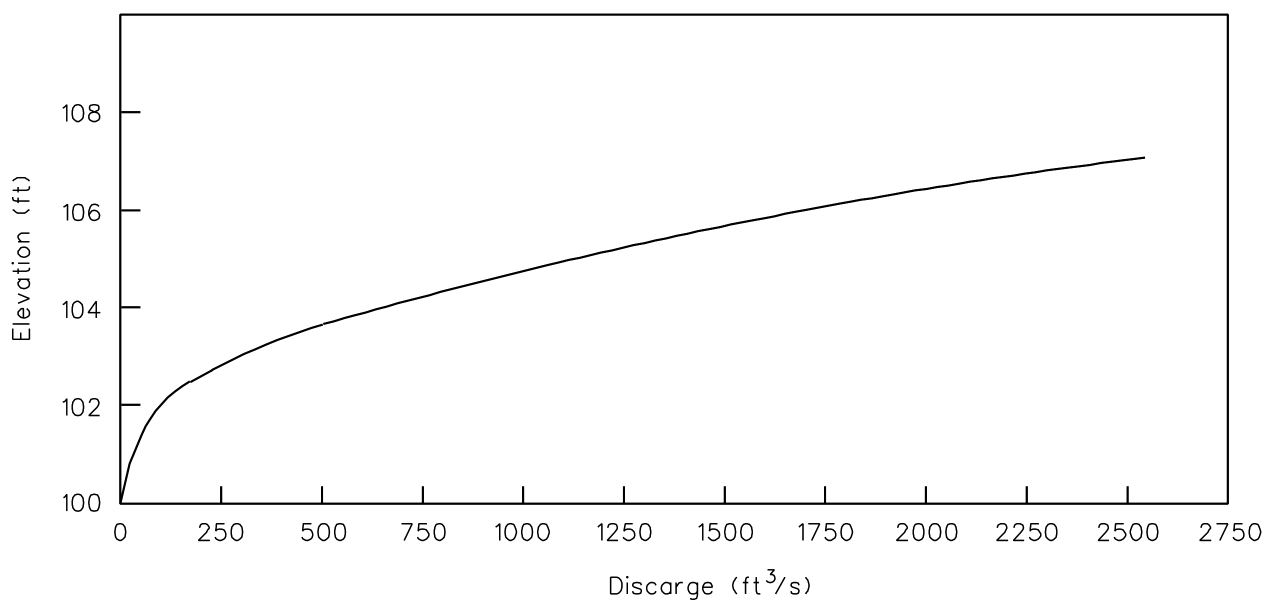

FIGURE 15.C-9 — Stage vs. Discharge

Using the minimum flow depth (stage) of 101 ft for fish migration as agreed upon in Step 1, Figure 15.C-9 can now also be used to estimate the stage threshold corresponding to the previously computed "mean annual" minimum discharge for the October low-flow period for migration in the natural channel. This discharge was found to be 0.62 m^3/s and is plotted on Figure 15.C-6.

Step 6 MIGRATION PERIOD. Next, determine the migration periods when the design fish can move up the natural channel to reach the culvert. Consider both the high- and low-flow period.

High-Flow Period. In Step 5, the foregoing maximum discharge threshold of 3.54 m^3/s was plotted on Figure 15.C-5. As a result, it is now possible to roughly estimate the potential number of days that adult fish might be able to migrate through the reach during maximum flow periods to even reach the culvert. The culvert will be designed to try and perpetuate at least a portion of these estimated migration days. Using this procedure, it was estimated that adult fish could conceivably have a time distribution of high-flow migration consisting of 48 d as shown in Table 15.C-8. Again, this is for the natural channel and provides some indication as to how often migrating fish can be expected to reach the culvert.

TABLE 15.C-8 — Natural High-Flow Migration Days

May 1-10	May 11-20	May 21-31	June 1-10	June 11-20	June 21-30
10	4	4	10	10	10

Low-Flow Period. Plot the minimum discharge of 22 ft^3/s from the foregoing Step on Figure 15.C-6. From this plot, it is now possible to roughly estimate the potential number of days that adult fish might be able to migrate through the reach during minimum flow days to even reach the culvert. As with high-flow days, the culvert would be designed to try and perpetuate this number of low flow days to some degree. From Figure 15.C-6, it was estimated that adult fish could probably have a time distribution of low-flow migration of 4 days as shown in Table 15.C-9.

TABLE 15.C-9 — Natural Low-Flow Migration Days

Oct 1-10	Oct 11-20	Oct 21-31
0	0	4

These findings in Table 15.C-8 and Table 15.C-9 were discussed with the responsible resource and regulatory agency(ies). It was pointed out that this is the expected natural (no highway) condition. The agency(ies) agreed to the findings for the spring migration, but insisted that they had "observed" October migration at flows of 5 ft^3/s. This results in a migration period of 30 d.

Step 7 TRIAL CULVERT GEOMETRY. The next step is to try and devise a culvert geometry that will tend to perpetuate these migration periods to the extent practicable. From the culvert analysis (see Culverts Chapter), a culvert size and type was selected that is compatible with the flood hazard to the road and adjacent property. Hint, if a multi-barrel culvert is expected to be required:

- ensure that the flowline of at least one barrel (termed here as the low-flow barrel) is at the stream profile elevation to serve as both a fish and flood passage barrel;

- all or some of the remaining flood passage barrels (termed raised barrels) are at a higher elevation than the low-flow barrel's inlet elevation (for the initial trial, place them slightly lower than the two-year flood stage, which should be approximately the low-flow or dominant channel's actual top of bank* elevation through the reach; and

- decide whether to place the outlet flowlines of the raised barrels at the same elevation as the (1) low flow barrel(s) to avoid bank headcutting during flows in excess of the bank full stage, which results in a steeper culvert slope in the raised barrel(s) than in the low-flow barrel(s), and/or (2) at the low-flow or dominate channel's actual top of bank*, which complicates the outlet protection necessary to avoid headcutting of the adjacent flood plain, but simplifies the structure geometry and hydraulic culvert analysis.

* If the channel is incised, use the theoretical mean annual or dominate channel top-of-bank elevation.

At this site it was determined, using the procedures in the Culverts Chapter, that a long, two-barrel culvert with the low-flow barrel being 60 in in diameter, and the raised barrel also being 60 in in diameter, would provide adequate flood protection to the highway and adjacent property and simplify the hydraulic analysis, structural design and detailing. This analysis also provided culvert performance curves for discharge versus both flow depth and velocity for the culvert. These findings are summarized in Table 15.C-10, Table 15.C-11 and Table 15.C-12. Only with high flows are both barrels functioning.

TABLE 15.C-10 — Performance Data Within Culvert
(Minimum Depth)

High Flows		Low Flows	
Depth (ft)	Velocity (ft/s)	Depth (ft)	Velocity (ft/s)
3.0	4.5	1.1	3.9

The high-flow velocity performance curve coupled with the sustained fish speed of 4.0 ft/s and darting speed of 10.0 ft/s resulted in an estimate of 150 ft^3/s above to migration would probably not occur through either culvert barrel in May and June (academic due to constraints imposed by the channel hydraulics). Similarly, the performance curve for discharge versus depth indicated that, below a discharge of 10.ft^3/s, migration would probably not occur through the low-flow culvert barrel in October. During these two periods, the fish would probably not be able to even reach the culvert or migrate within the channel exclusive of any culvert due to the natural velocity (May-June) or depth (October) constraints.

TABLE 15.C-11 — Performance Data at Culvert Outlet
(Brink or Critical Depth)

High Flows		Low Flows	
Depth (ft)	Velocity (ft/s)	Depth (ft)	Velocity (ft/s)
2.0	9.3	0.7	4.1

TABLE 15.C-12 — Performance Data at Culvert Entrance
(Critical Depth)

High Flows		Low Flows	
Depth (ft)	Velocity (ft/s)	Depth (ft)	Velocity (ft/s)
2.2	9.0	0.8	2.3

Step 8 FISH PASSAGE. It is now necessary to determine if the trial culvert geometry will provide an acceptable fish passage. Hopefully this trial culvert geometry might provide for a reasonable, even though reduced, number of high-flow (May-June) and low-flow (October) migration days to not adversely interfere with the predicted natural migration periods of Figure 15.C-5 and Figure 15.C-6.

High-Flow Period. The lower barrel is on a slope of 0.5%, and in Step 5 it was determined that migration may occur in the channel up to a stage of 101 ft, to corresponds to a discharge of 125 ft^3/s and velocity of 4.0 ft/s. Recall that above this discharge and corresponding stage, the higher velocities would probably limit natural migration in the channel.

The flow velocity in the low-flow, fish passage barrel was determined from the velocity-discharge performance curve for just that barrel obtained from a culvert analysis based on the two barrels. From this performance curve, it was determined that the culvert velocity for the low-flow barrel would exceed 4.0 ft/s when the total runoff discharge of 50 ft^3/s reaches the culvert. By plotting this discharge on Figure 15.C-5, 36 migration days occur as shown in Table 15.C-13.

TABLE 15.C-13 — Culvert High-Flow Migration Days

May 1-10	May 11-20	May 21-31	June 1-10	June 11-20	June 21-30
10	3	0	3	10	10

Low-Flow Period. Similar to the foregoing approach used for high flows, estimate the depth through the low-flow culvert barrel for the mean annual minimum migration discharge of 22 ft^3/s (as estimated in Step 2) for the low-flow period. Recall that, at lower discharges, natural migration would not be expected (at least not by the Department). In this Example, the depth in the lower, fish passage barrel was determined from the depth-discharge performance curve as obtained from the culvert analysis based on both barrels. From the culvert performance curve, it was also determined that the depth would drop below the required 1 ft at a discharge of 50 ft^3/s. By plotting this discharge on Figure 15.C-6, 19 culvert migration days are found as shown in Table 15.C-14.

TABLE 15.C-14 — Culvert Low-Flow Migration Days

Oct 1-10	Oct 11-20	Oct 21-31
0	9	10

After comparing Table 15.C-8 with Table 15.C-13 and Table 15.C-9 with Table 15.C-14, it was determined that the total (both high- and low-flow periods) number of migration days would probably decrease from 48 to 36 d in the Spring and 30 to 19 d in October. As such, it was decided by the responsible resource and regulatory agency(ies) that any culvert alternative was unacceptable, as proposed, to meet fish migration needs. Although the Deaprtment believed that this was an unreasonable ruling, further negotiations proved to be pointless.

Given these findings, consider Example 3 for constructing a downstream sill(s) with a notched weir crest to obtain a greater low-flow culvert flow depth, lower the culvert velocity and increase the number of days where fish migration could occur.

15.C.3 EXAMPLE 3 — DOWNSTREAM SILL DESIGN

Assume you have a site where unsatisfactory low-flow performance occurred with a smooth culvert (Example 2) or with an unnotched sill. As such, it is decided to try a notched sill:

Step 1 DISCHARGES AND CRITERIA. The input and geometries are from Example 2. First, devise a trial notched sill geometry using one or more downstream sills to accommodate the three discharges determined in Example 2 and the selected sill criteria.

Discharges. Consider the three discharges:

- Minimum migration discharge (October) = 22 ft^3/s.
- Maximum migration discharge (May-June) = 125 ft^3/s.
- Design flood discharge for culvert (Q_{50}) = 500 ft^3/s.

Sill Criteria. The sill and culvert geometry for these three discharges must satisfy the following criteria for the design fish of Example 2 (repeated from Step 1 of that Example):

- maximum velocity of 4.0 ft/s or less inside the culvert,

- minimum depth of approximately 1 ft or more inside the culvert and at the inlet and outlet,
- culvert inlet and outlet velocity of 10 ft/s or less (design fishes' darting speed),
- sill brink (burst) velocity of 10 ft/s or less, and
- no jump height greater than 0.5 ft.

Step 2 SILL GEOMETRY. Select the trial geometry for the sill and notch, and determine if the notch geometry will be adequate for the low-flow period. Six substeps are suggested:

- Select trial sill height.
- Determine trial notch depth.
- Determine maximum allowable head on sill.
- Determine notch width.
- Check low-flow fish passage of notch and fish jump height.

Select Trial Sill Height, $P_{S.}$ Assume one downstream sill will be sufficient. Select a rectangular-shaped notch. Referring to Section 15.4.10, the minimum sill height, P_S, above the streambed is determined based on passing the entire low-flow minimum migration discharge (October) of 22 ft^3/s from Example 2 through the notch. Water surface profiles are computed through the culvert using this low-flow discharge and different trial head water depths on the sill. From this, it is determined that if the total sill height, P, must be at least 3 ft for the maximum velocity within the culvert to equal or be less than 4.0 ft/s, and for the inlet and outlet velocity to be equal to or less than 10 ft/s. Also, it was determined that, with the trial P_S = 2 ft, the flow depths through the culvert (outlet, interior, inlet) will be equal to or greater than approximately 1 ft, and a hydraulic jump will not occur.

Determine Trial Notch Depth, P_n. The notch in the sill must pass the mean annual minimum migration discharge of 22 ft^3/s (October) in compliance with the above criteria. Again, referring to Section 15.3.10, try a notch that is $P_n = H_n$ = 1 ft deep. Thus, the notch crest is located $P_S = P - H_n$ or 3 – 1 = 2 ft above the streambed (Figure 15.C-10).

Determine Maximum Allowable Head, P_n. The trial culvert's performance curve of tailwater (TW) versus discharge (Q) from Example 2 is used to estimate tailwater depths (see Culverts Chapter). From this figure, it is determined that for the 10-yr culvert design discharge of 500 ft^3/s, the maximum tailwater before an unacceptable flood hazard occurs is 3.3 ft. Thus, the maximum head plus total sill height will be H + P + (S_oL_1) = 6.6 ft, which is the maximum tailwater (given a decreasing channel profile elevation in the downstream direction) where S_o is the channel slope, and L_1 is the distance between the culvert and the sill. From the Culverts Chapter, the length of the expected scour hole at the culvert's 10-yr design discharge of 500 ft^3/s is 65 ft. This means that the sill must be at least this far downstream to avoid damage from culvert scour. Therefore, let L_1 = 70 ft.

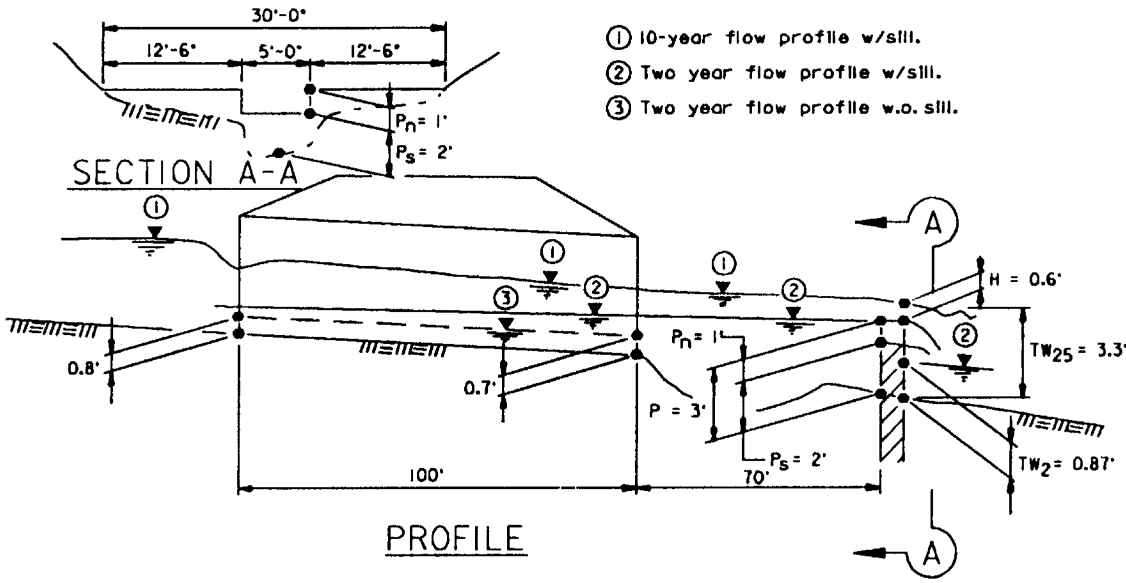

FIGURE 15.C-10 — Sill Geometry

Based on these findings, the maximum allowable head for the 10-yr flood plus the total sill height for the sill is $H + P = TW + (S_o)(L_1)$, or the maximum allowable head on the sill (not the notch) is $H = TW + S_oL_1 - P = 3.3 + 0.3 - 3 = 0.6$ ft.

Determine Notch Width, L_n. Because the mean annual minimum discharge for October of $Q_n = 22$ ft^3/s is to be confined to the notch, the low-flow head on the notch cannot exceed the foregoing trial notch depth of, again, $P_n = H_n = 1$ ft.

Assume (and verify later) that the weir will be sharp crested and not submerged. From Section 15.4.10, the weir equation for a rectangular notch is:

$$Q_n = C_1 L_{EN} H_{EN}^{1.5}$$

Here, L_{EN} is the effective weir crest and H_{EN} is the effective weir head. Assume that the active approach channel width for the notch width, b_n, is the same as the assumed notch width, (L_n) (S_0), by assuming $L_n = b_n$:

$$H_n/P_n = 1/1 = 1$$

Try a low-flow weir crest (notch length) $L_n = 5$ ft:

$$L_n/b_n = 5/5 = 1$$

From Figure 15-31b:

$$C_1 = 3.9$$

From the weir equation and for the October mean annual minimum migration discharge of $Q_n = 22$ ft^3/s:

$L_{EN} = Q_n/(CH_N^{1.5})$
$L_{EN} = 22/(3.9)\ (1.0^{1.5})$
$L_{EN} = 5.6$ ft

From Figure 15-31a:

$K_L = -\ 0.005$ or say 0.

Because the effective low-flow weir crest (notch length) is:

$L_{EN} = L_n + K_L$, then:

$L_n = L_{EN} - K_L = 5.6 - 0 = 5.6$ ft, which is essentially equal to the trial L_n.

As defined in Figure 15.C-10, the tailwater for 22 ft^3/s is approximately 0.9 ft. So:

$H_2 = TW - P_S = 0.9 - 2.0 = -\ 1.1$ ft.

Because H_2 is negative, the notch is not submerged by the 22 ft^3/s, and the initial assumption that the weir was not submerged is valid. Had this not been the case, then either:

- ensure that, if submerged, the head, H_n, is still less than the assumed notch depth, P_n;
- try and decrease the notch depth; or
- evaluate submerged or partially submerged conditions.*

* *This is done by trial and error; assume a weir (notch) length and compute Q_1 based on submerged or partially submerged conditions until Q_1 is equal to the low-flow migration discharge.*

Check Low-Flow Fish Passage Of Notch. Because the sill notch is not submerged, the brink depth will be equal to approximately $0.715d_c$. The critical depth for the rectangular notch of width, $b_n = 5$ ft, and discharge, Q_n, of 22 ft^3/s is:

$d_c = [(Q_n/b_n)^2/g]^{1/3}$
$d_c = [(22/5)^2/32.2]^{1/3} = 0.8$ ft

the brink depth, $d_{nb} = 0.715d_c = (0.715)(0.8) = 0.6$ ft, and

the notch brink velocity, $V_{nb} = Q_n/(b_n d_{nb}) = 0.22/((0.6)(5)) = 7.3$ ft/s.

Because the low-flow notch velocity, V_{nb}, is less than 10 ft/s and the brink depth is close to the required 1 ft, the head, H_n, on the notch is not greater than the notch depth, $P_n = 1$ ft; then the notch trial size of $L_n = 5$ ft wide and $H_n = 1$ ft deep with a distance from streambed to the notch crest of $P_S = 2$ ft is acceptable.

Check Fish Jump Height: The required height the design fish must jump is the downstream water surface elevation to approximately 1 ft above the notch crest. This is equal to P_S – TW for the low-flow migration discharge (October) or:

Required Jump Height: $2 - 0.9 = 1.1$ ft.

Because this required jump height is approximately equal to the selected criteria of 1 ft, the notch and sill geometry to this point are acceptable.

Step 3 TOTAL SILL CREST LENGTH. Having apparently satisfied the low-flow period requirements, attention must be directed to determining the total crest length necessary to convey the culvert design (and perhaps review) flood. To avoid lateral erosion and future bypassing of the sill, it will be necessary to provide armored dikes between the culvert outlet and the sill sufficient to contain the design (or perhaps review) flood. Refer to the Bank Protection Chapter for design practices.

In this Step, estimate:

- total sill crest length,
- discharges over the sill crest, and
- sill length.

Total Sill Crest Length. The total sill crest length (including the notch width) must pass the culvert's 10-yr design discharge of 500 ft^3/s at a head not to exceed the previously computed maximum allowable head for the sill of H = 0.6 ft. Also, this same sill should, if practicable, meet the foregoing fish migration criteria somewhere along the total sill crest (either in or outside the notch portion of the sill, or preferably both).

Discharges Over Sill Crest. The total maximum discharge over the sill crest is the sum of the discharges along the sill crest for the previously computed maximum allowable head; i.e., the sum of the discharges for the:

- sill with a trial of H = 0.6 ft exclusive of the notch width, and
- discharge along the notch width.

The discharge along the notched portion of the sill is computed as follows: For the notch portion, assume the approach flow width, b_n, for the culvert maximum design discharge is equal to the crest length, L_n = 5 ft, so that L_n/b = 1.0. The corresponding head on the notch crest would be the maximum allowable head plus the notch depth or:

$$H_{n\,MAX} = H + H_n = 0.6 + 1 = 1.6 \text{ ft}$$

The unsubmerged discharge, Q_1, corresponding to this head is:

$$Q_1 = C_1\, L_n H_{nMAX}^{1.5}$$

The distance from the streambed to the notch crest was P_S = 2 ft so that H_{nMAX}/P_S = 1.25. With L_n/b = 1.0, C_1 from Figure 15-31b is 3.7. Also, with L_n/b = 1, from Figure 15-31a, K_L = - 0.005:

$$Q_1 = (1.0)(5)(1.6^{1.5}) = 10.1 \text{ ft}^3/\text{s}$$

The distance between the tailwater corresponding to the maximum culvert design discharge and the maximum allowable head is H_2 or:

$$H_2 = \text{Tailwater} - P$$
$$H_2 = 3.3 - 2 = 1.3 \text{ ft}$$

Therefore:

$$(H_2/H_1)^{2.5} = (1.3/1.6)^{2.5} = 0.60$$

From Curve 3 on Figure 15-32:

$$Q/Q_1 = 0.68$$

The notch at this flow rate is submerged; therefore, the maximum discharge passing over the notch portion of the sill is:

$$Q_{nMAX} = Q = (0.68)(Q_1) = (0.68)(10.1) = 7 \text{ ft}^3/\text{s}.$$

This means that the unnotched portion of the sill must convey the remaining portion of the maximum culvert design discharge, Q_{10}, of 500 ft^3/s, or:

$$Q_{10} = 500 - Q_{nMAX} = 500 - 7 = 493 \text{ ft}^3/\text{s}$$

Sill Length. Try a sill length, L_s, (minus notched portion of L_n = 5 ft) equal to 25 ft, and the previously computed maximum allowable head, H = 0.6 ft.

From the downstream cross section of Example 2, the approach channel width at the maximum allowable head of P + H – TW = 3 + 0.6 – 3.3 = 0.3 ft, which is approximately b_s = 25 ft (30 ft – 5 ft notch).

With the previously computed height from streambed to top of sill and a trial L_s = 25 ft:

$$L_s/b_s = 25/25 = 1.0$$
$$H/P_S = 0.6 / 3 = 0.2$$

Then, from Figure 15-31b:

$$C_1 = 3.3$$

With L_s/b_s = 1.0, from Figure 15-31a, K_L = – 0.005 or say 0.
From the weir equation:

$L_s = (Q_{10} - Q_{nMAX})/C_1H^{1.5} = (500 - 7)/((3.3)(0.6^{1.5})) = 321$ ft

A sill crest of this length is impractical and clearly does not fit the natural channel. Raise the roadway gradeline 3 ft. The previous calculation for Q_{nMAX} results in a value of 34 ft^3/s so that $(Q_{10} - Q_{nMAX}) = 500 - 34 = 466$ ft^3/s. L_s now equals:

$L_s = 466/((3.3)(3.6^{1.5})) = 20.7$ ft

The tailwater for the Q = 500 ft^3/s is 3.3 ft (Figure 15.C-10). This is $TW - P_S = 3.3 - 3 = 0.3$ ft above the sill crest. See if this degree of submergence affects the crest length.

From above, $H_1 = 3.6$ ft:

$H_2 = H + P_S - TW = 3.6 + 3.0 - 3.3 = 3.3$ ft
$(H_2/H_1)^{2.5} = (3.3/3.6)^{2.5} = 0.8$ ft

Step 4 HIGH FLOW MIGRATION CHECK. From Figure 15-31, C_1 still equals 3.3, so this amount of submergence had no effect.

The sill length + notch width of 22 ft + 5 ft = 27 ft is acceptable because it will convey the $Q_{10} = 500$ ft^3/s without causing a flood hazard given the foregoing 3-ft gradeline raise and no upstream property.

To this point, the design apparently satisfies the criteria for the:

- low-flow (October) migration discharges, and
- 10-yr culvert design flood to avoid a flood hazard.

Consideration can now be given to fish migration during the mean annual high-flow period (May-June). The discharge for this period when fish can physically transit the channel is, from Figure 15.C-5, 125 ft^3/s.

Evaluate the performance of this sill for the maximum migration discharge (May - June) of 125 ft^3/s. This discharge will be passing both above the notch and over the unnotched sill crest. Clearly, the elevation of the head will be the same for both. The question now is how would this migration discharge be divided between the notch crest and the sill crest? This is found by plotting a performance curve of head vs. discharge for both the notch and the unnotched sill. The desired value of H is identified where the trial value of H results in a discharge from each curve whose sum is the maximum migration discharge. Using the weir equation $Q = C_1LH^{3/2}$, and Figure 15-31 and Figure 15-32, the plot of Figure 15.C-11 was prepared (Note: $b_n = L_n$).

From Figure 15.C-11, it is estimated that the head for the mean annual maximum migration discharge (May - June) of 125 ft^3/s is approximately 1.3 ft.

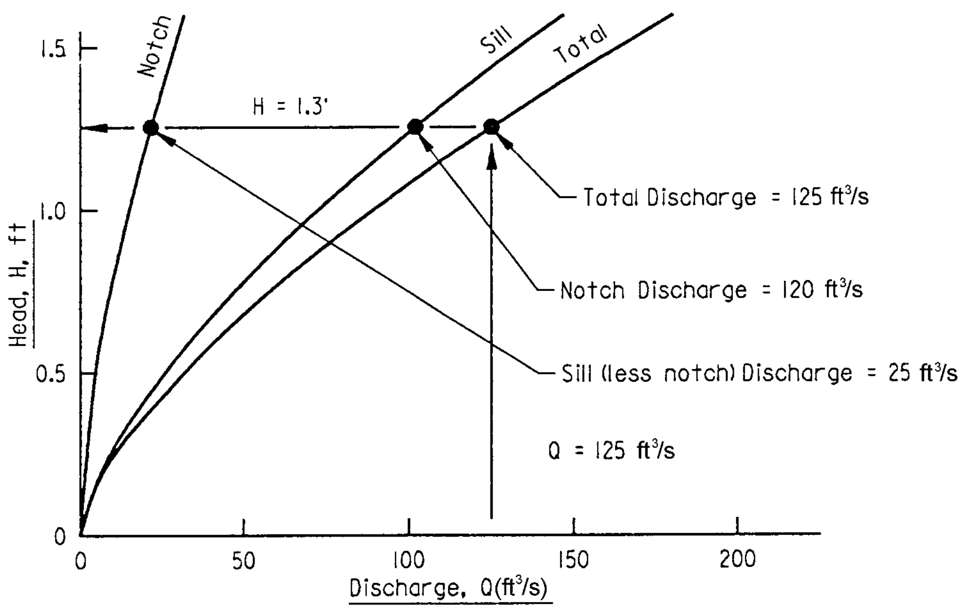

FIGURE 15.E-11 — Sill and Notch Performance Curves
(Maximum Migration Discharge)

The notch is not submerged by the maximum low-flow discharge tailwater of 1.9 ft (i.e., $P_S - TW = 2 - 1.9 = + 0.1$ ft). For the notch discharge from Figure 15.E-1, the minimum discharge is 25 ft^3/s. Critical depth through the notch for this discharge is:

$$d_c = [(Q/b)/g]^{1/3} = [(25/5)/32.2]^{1/3} = 0.54 \text{ ft}$$

Because the notch opening is not submerged, then the overflow depth is $0.715d_c = 0.715(0.54) = 0.39$ ft. The corresponding velocity would be:

$$V = Q/(0.715d_cL_n) = 25/((5)(0.39)) = 12.8 \text{ ft/s}$$

Because this velocity is slightly larger than 10 ft/s, there is some question on whether fish can migrate through the notch. However, because the notch is not submerged, fish must jump over the notch sill anyway to move upstream. Assuming critical depth at the upstream face of the notch where a jumping fish might land, the velocity would then be:

$$V = Q/(d_cL_n) = 25/((5)(0.54)) = 9.3 \text{ ft/s}$$

Because 9.3 ft/s is less than 10 ft/s, fish should be able to jump through the notch during the May-June mean annual runoff period when fish can transit the channel to reach the culvert (< 125 ft^3/s; Figure 15.C-5).

Next, although not critical in this Example, for illustration purposes determine if fish can migrate over the remaining sill crest (beyond notch) by jumping.

The maximum migration discharge jump height is P − TW = 3 − 1.9 = 1.1 ft, which is approximately equal to the 1-ft jump criteria; as such, most fish should be able to jump over the sill crest beyond the notch if not precluded by the brink depth or velocity on this part of the sill crest.

A depth of 0.715 critical depth will probably occur at the brink because the tailwater is below the sill crest. The critical depth for the 120 ft^3/s discharge is:

$$d_c = [(Q/L_s)/g]^{1/3}$$

$$d_c = [(120/22)/32.2]^{1/3} = 0.55 \text{ ft}$$

Because there is only 0.3 ft of submergence, assume that the brink depth is:

$$d_b = 0.715d_c = (0.715)(0.55) = 0.4 \text{ ft}$$

Because 0.4 ft is greater than the 0.3 ft of submergence, the brink velocity is approximately:

$$V_b = Q/(d_bL_s) = 120 /((0.4)(22)) = 13.6 \text{ ft/s}$$

This velocity is greater than 10 ft/s. Again, however, if the velocity upstream where jumping fish might land after jumping is that associated with say d_c = 0.55 ft, then the velocity would be 120/((0.55)(22)) = 9.9 ft/s. This is still greater than 10 ft/s. However, because they can pass through the notch, which was designed for fish passage during the mean annual discharge, this is not a critical concern in this Example.

Although generally not pursued in detail in this Example, the designer should be aware of the following items:

- If a suitable sill and notch geometry could not be devised, it may be necessary to use two or more sills.
- It is essential that the sill ends (abutments) be secure from erosion/scour and thus being bypassed by very large floods that are in excess of the culvert design flood selected to avoid a flood hazard to adjacent property and/or the traveling public.
- A stable scour hole downstream of the sill and culvert is needed and can be designed using the technology in the Energy Dissipator Chapter.

15.C.4 EXAMPLE 4 — COUNTERSUNK CULVERT FISHWAY

Use the complex hydrology findings of Example 2 and its resultant multiple barrels to illustrate the ramifications involved in countersinking one barrel:

Steps 1-6 DISCHARGE AND CRITERIA. Clearly the criteria, hydrology and site geometry must first be determined. Using the findings from Steps 1 through 6 of Example 2, investigate the countersunk culvert alternative to try and satisfy fish passage requirements.

Step 7 TRIAL CULVERT GEOMETRY. Next, select a trial culvert geometry keeping in mind that it will be partially buried (countersunk) below the streambed; i.e., it will not have all the waterway opening that might be expected in a conventional culvert analysis. The designer will have to make allowances for this modified culvert geometry (the "any shape" feature of HY-8 of the HYDRAIN system is suggested; see Culverts Chapter).

Using the insights provided by the analysis of Example 2, it is expected that the same geometry will continue to satisfy the Department's flood hazard policy, and also serve as a reasonable trial culvert geometry to satisfy fish migration needs. (Note: the friction factor of the countersunk barrel should reflect the expected material to be used for substrate in the countersunk portion of the culvert. See discussion in Section 15.E.1.3). However, the lower barrel must now be 84 in in diameter to allow for being countersunk 2 ft. The raised barrel will remain 60 in in diameter.

The velocity in the lower, fish-passage barrel was again determined from the velocity-discharge performance curve for just that barrel as obtained from a culvert analysis based on both barrels. As before, from this performance curve, it was determined that the culvert velocity would exceed 4 ft/s at approximately a discharge of 50 ft^3/s (remembering from Figure 15.C-5, fish can migrate in the natural channel until the velocity exceeds 125 ft^3/s). By plotting this discharge on Figure 15.C-5 of Example 2, the number of migration periods and durations are found. These will remain the same as before and are shown in Table 15.C-13.

Step 8 SUBSTRATE STABILITY. Because this is a countersunk culvert, the riprap to be placed in the culvert should withstand the expected interior culvert velocities for some recurrence interval acceptable to the responsible resource and regulatory agency(ies). Through negotiation with these responsible resource and regulatory agency(ies), a discharge corresponding to 125 ft^3/s was selected as the discharge for evaluating the stability of the riprap to be placed inside the culvert for substrate. This selection presumes that a flood equal to or greater than 125 ft^3/s would "flush" the riprap out of the culvert. Clearly, this would occur annually as the mean annual discharge from Example 2 was 455 ft^3/s. However, it might also be assumed that, when this occurs, bed load may be transported into the culvert from upstream and, thus, at least partially replace the lost riprap. (The Culverts Chapter addresses this type of analysis in evaluating culvert deposition problems). Also, by countersinking the culvert floor 2 ft below streambed, additional waterway opening will be obtained should the riprap substrate be "flushed" out and not replaced by the bed load. This would provide additional waterway opening thereby reducing the culvert velocity and increasing the flow depth to where the loss of this riprap may not pose a depth or velocity limiting fish migration problem. This will be evaluated later, but it should be resolved with the responsible resource and regulatory agency(ies) now.

For simplicity, use tractive shear to estimate the stability of the riprap to be placed in the culvert; see Channels Chapter. From the culvert analysis, the flow depth, velocity and flow area in the barrel corresponding to 125 ft^3/s is:

$d = 2.5$ ft, $V = 6.3$ ft/s, and $A = 18.7$ ft^2

A rough estimate of the water surface slope (unless a better estimate is available from a water surface profile analysis through the culvert) is, from Manning's equation, $S = [(Vn)/(1.486)(R^{2/3})]^2$.

$R = A/P = 18.7/13.4 = 1.4$, or

$R^{2/3} = 1.4^{2/3} = 1.25$ so that

$S = [(6.7)(0.035)/(1.486)(1.25)]^2 = 0.016$ ft/ft

The actual tractive shear for this flow depth would be approximately:

$\tau = \gamma RS$
$\tau = (62.4)(1.4)(0.016) = 1.4$ lbs/ft^2

From the Channels Chapter, the required D_{50} stone to avoid displacement would be approximately 3-in to 6-in equivalent stone diameter or larger for a velocity of 6.7 ft/s. Ignoring velocity but considering a tractive shear of 1.4 lbs/ft^2 results in a required stone D_{50} of approximately 1.5 in. A 6-in stone or larger should be quite adequate.

Should the riprap for some reason be displaced out of the culvert by the discharge of 125 ft^3/s, the design fish might still transit the culvert in that the flow depth within the culvert would be increased by as much as 2 ft and provided that the velocity within the culvert is 4 ft/s or less. Low-flow migration through the culvert would also be enhanced by the increase in flow depth. However, the 6.7 ft/s, which exceeds the 4-ft/s criteria, would be reduced to 4.5 ft/s. Because this velocity exceeds the 4-ft/s threshold, then it is necessary to:

- revise the lower culvert barrel geometry,
- negotiate new criteria, or
- select a different fish-passage alternative.

Step 9 FISH PASSAGE. If the velocity had been equal to or less than 4 ft/s, then the next Step is to see if there are sufficient migration periods and durations as with Example 2. Then, check and see if this trial culvert geometry might provide for a reasonable number of high-flow (May-June) and low-flow (October) migration periods and durations to not adversely interfere with the projected natural migration periods shown on Figure 15.C-5, Figure 15.C-6, Table 15.C-10 and Table 15.C-11 of Example 2.

15.C.5 EXAMPLE 5 — OPEN BOTTOM CULVERT FISHWAY

This site has a deep, incised channel with competent-appearing foundations thereby making this type of fish-passage facility feasible. Through negotiations with the responsible resource and regulatory agency(ies), it was agreed that the simple analysis for estimating an open-bottom culvert size to accommodate fish migration would suffice. The following computations illustrate the hydraulics of a bottomless structural plate arch using a simple analysis:

Step 1 CRITERIA. Same as Example 1.

Step 2 DATA AND HYDROLOGY. The necessary site data is obtained and the hydrology is estimated:

Average Stream Slope = 0.02 ft/ft

Q_{25} = 755 ft^3/s, Q_{10} = 500 ft^3/s, Q_2 = 300 ft^3/s

Normal summer flow, Q_s, is between 15 and 25 ft^3/s

To satisfy highway criteria, HW/D < 1 for Q_{25} (see Example 1):

$Q_1 = (0.65)(Q_{10}) = (0.65)(500) = 325\ ft^3/s$
$Q_2 = (0.2)(Q_{10}) = (0.2)(500) = 100\ ft^3/s$

For the simple method, the discharges and velocities to be used in evaluating the fish passage are:

V_1 = 5 ft/s for 65% of the 10-yr discharge
V_2 = 3 ft/s for 20% of the 10-yr discharge

Step 3 TRIAL CULVERT. For HW/D $\leq$ 1, projecting entrance, Q_{25} = 755 ft^3/s and inlet control, an open-bottom structural plate arch having a 20-ft span and a 8 ft- 3½ in rise is selected as meeting the highway criteria; see Culverts Chapter.

Step 4 FLOOD STAGE CHECK. Criteria for an open-bottom type structure requires that it be considered only in channels that are:

- stable,
- incised with no significant overbank flows,
- scour-limited by streambed material, or
- founded in scour-resistant bedrock.

The natural channel has approximately a 10-ft bottom and banks sloping at approximately 1.4 ± horizontal to 1.0 vertical. Notably, floods can escape the banks only at approximately a depth of 4.5 ft. Check the flow depth in the channel for Q_{25} = 755 ft^3/s. The Manning's n value is approximately 0.035. Assuming a flow depth = 4.5 ft, the approximate wetted perimeter is P = 10 + (2)(6.4) = 23 ft. The approximate flow area is A = (14.5)(4.5) = 65 ft^2. The hydraulic radius is R = A/P = 65/23 = 2.83 ft. Using Manning's equation, $Q = 1.486/nR^{2/3}S^{1/2}A$, $Q = (1.486)/(0.035)(2.83)^{2/3}(0.02)^{1/2}(65)$ = 781 ft^3/s > 755 ft^3/s; therefore, the design is satisfactory because the flow depth is less than 4.5 ft; i.e., the design flood will be within the normal, incised channel cross section. As such, the arch spans the flood channel and thus meets the above criteria. Using the practices in the Channels Chapter, the channel is determined to be stable thereby meeting the other criteria.

Step 5 APPROVALS. In negotiations with the responsible resource and regulatory agency(ies) it was pointed out that the culvert cross section is essentially the same width as the natural channel; therefore, if fish can migrate upstream given the downstream channel conditions, their movement should not be impaired by this culvert. However, should the natural stream condition block part of a fish run at or near the culvert site, the Deaprtment may elect to construct a fishway that will allow free passage over this natural obstacle if the cost is modest. The responsible resource and regulatory agency agreed.

Step 6 SCOUR HAZARD. In accord with the third criteria in Step 4, cause for concern in designing a bottomless arch is the stability of the bed and the material surrounding the footings of the arch. This type of structure commonly fails due to undermining of the footings. If much overbank flow occurs, the waterway will be significantly constricted by the culvert thereby resulting in even higher velocities at the inlet and outlet and through the culvert. Even when the flood is not constricted, the bed may become mobile so that scour occurs. If significant quantities of bed material are unstable under these severe flow conditions, serious degradation within the confines of the arch can occur depending upon the bed material and underlying foundation material. This may leave the arch's footings in a vulnerable position and result in a steeper, undesirable fish migration channel through the culvert. For these reasons, a liberal factor of safety should be used when designing the foundations for open-bottom culverts.

A tractive shear stability analysis of the bed for the design flow conditions should always be made at a minimum to ensure substrate stability during the culvert's design flood. At sensitive sites, a more thorough analysis such as provided by a mobile bed computer models such as BRI-STARS, HEC-6 or FLUVIAL can be used.

15.C.6 EXAMPLE 6 — SILL/BAFFLE CULVERT FISHWAY

Previous Examples illustrated the simple and complex hydrology methods and three types of fish-passage designs. This Example will illustrate the development of a culvert performance curve and its use in another type of fishway design. It is assumed that the sill or baffle geometry has been checked and found to be satisfactory for the selected range of fish migration discharges; i.e., the brink depths and velocities and the pool velocities and any jumping criteria were met.

This Example will discuss baffles that are:

- full width of a culvert (Steps 1-3),
- less than full width of a culvert (Step 4), and
- located at the outlet (Step 4).

Full-Width Baffle

Step 1 TRIAL CULVERT GEOMETRY. Consider a reinforced concrete box culvert with the following dimensions and geometry:

Span (B) = 8 ft
Rise (D) = 8 ft
Culvert Length (L_a) = 87 ft
Culvert Slope (S_o) = 0.02 ft/ft

At inlet, wingwalls are 12 ft

Use Full-Width Baffles (reference Figures 15-13 through 15-18).

Baffle Height (h) = 1.7 ft, Length (l) = 8 ft.

Baffle Spacing (λ) = 9 ft with 2-ft long by 8-in deep, alternating notches. Assume that there is no baffle at the outlet end.

Step 2 HEAD V. DISCHARGE. Neglect the effect of the alternating notch on the resistance coefficient. Therefore:

$\lambda/D = 9/8 = 1.12$
$h/D = 1.7/8 = 0.21$

From Figure 15-13, f = 0.23.
(For comparison, where n = 0.050, f = 0.0926).

From the Culverts Chapter, the equation for computing the headwater for outlet control is:

$[1 + k_e + (fL_a/4R_H)]V^2/2g$

$k_e = 0.4$ for wingwalls at 30^o to 75^o with a square-top edge

$L_a = 87$ ft

$R_H = A/P = 64/32 = 2$ ft

$H = [1 + 0.4 + (0.23)(87)/(4)(2)]\ V^2/2g = 3.9\ V^2/2g$

Using this equation, complete Table 15.C-15 for a range of discharges.

TABLE 15.E-15 — Discharge vs. Head

Q (ft^3/s)	A(Full) (ft^2)	V = Q/A (ft/s)	$V^2/2g$ (ft/s^2)	H = 3.9 $V^2/2g$ (ft)
353	64	5.5	0.5	2.0
706	64	11.0	1.9	7.4
1059	64	16.5	4.2	16.4
1412	64	22.0	7.5	29.3

Step 3 PERFORMANCE CURVE. Performance curves for a culvert with a fishway can be estimated using the procedures in the Culverts Chapter and elsewhere in this Chapter. Using an assumed invert elevation of 100 ft results in performance curves similar to those in Figure 15.C-12. Note that over the range of discharges from 353 ft^3/s to 1059 ft^3/s, the culvert operates exclusively in outlet control.

In addition, the outlet control curve for the same barrel without fish baffles (n = 0.012), and the outlet control curve calculated by neglecting the cross sectional area of the baffles, are shown. For the smooth concrete barrel, the culvert operates in inlet control for the entire range of discharges. Thus, the fish baffles have transformed this short culvert from an inlet control structure to an outlet control structure — a potential flood hazard situation.

Notice also that simply neglecting the baffle area does not account for the energy losses due to the turbulence generated by the fish baffles. For example, at a discharge of 1059 ft^3/s, there is a headwater elevation difference of approximately 3 ft between the two curves; this difference would be greater for a longer culvert.

Step 4 OTHER BAFFLE CONFIGURATIONS. The following illustrates how to evaluate the other two baffle geometries. Specifically, baffles that are:

- less than the full width, and
- at the outlet.

Less Than Full-Width Baffle

The full-width baffle alternative was addressed in Steps 1-3. Adjustments are needed where the baffle is less than the full width of the barrel. Select a 2-ft wide notch for the full depth of the baffle, so that the baffles are now 8 – 2 = 6 ft long. Select a spacing of 9 ft. It is necessary to adjust the baffle spacing as previously discussed (do not adjust the baffle height).

Thus: λ/D (corrected) = (λ/D) (actual))(B/l)
$\lambda/D = (9/8)(8/6) = 1.5$

Then, from Figure 15-13, for $\lambda/D = 1.5$ and $h/D = 0.21$, $f = 0.25$. The differences in f could be greater at other ranges of λ/D.

Baffle at Outlet

To illustrate the effect of a baffle placed at the culvert outlet, assume that this was the case in the preceding example (Steps 1-3); then, $h_o = (d_c + h + D)/2$ or TW, whichever is larger.

Using outlet control calculations (reference Culverts Chapter), the computed values shown in Table 15.C-15 for h_o would apply for h = 1.7 ft. Similar to the preceding, discharge vs. brink depth is completed. These findings are shown in Table 15.C-16.

Note, if the baffles with the rounded top edge had been used, with $\lambda/D = 1.12$ and $h/D = 0.21$, from Figure 15-16, $f = 0.08$.

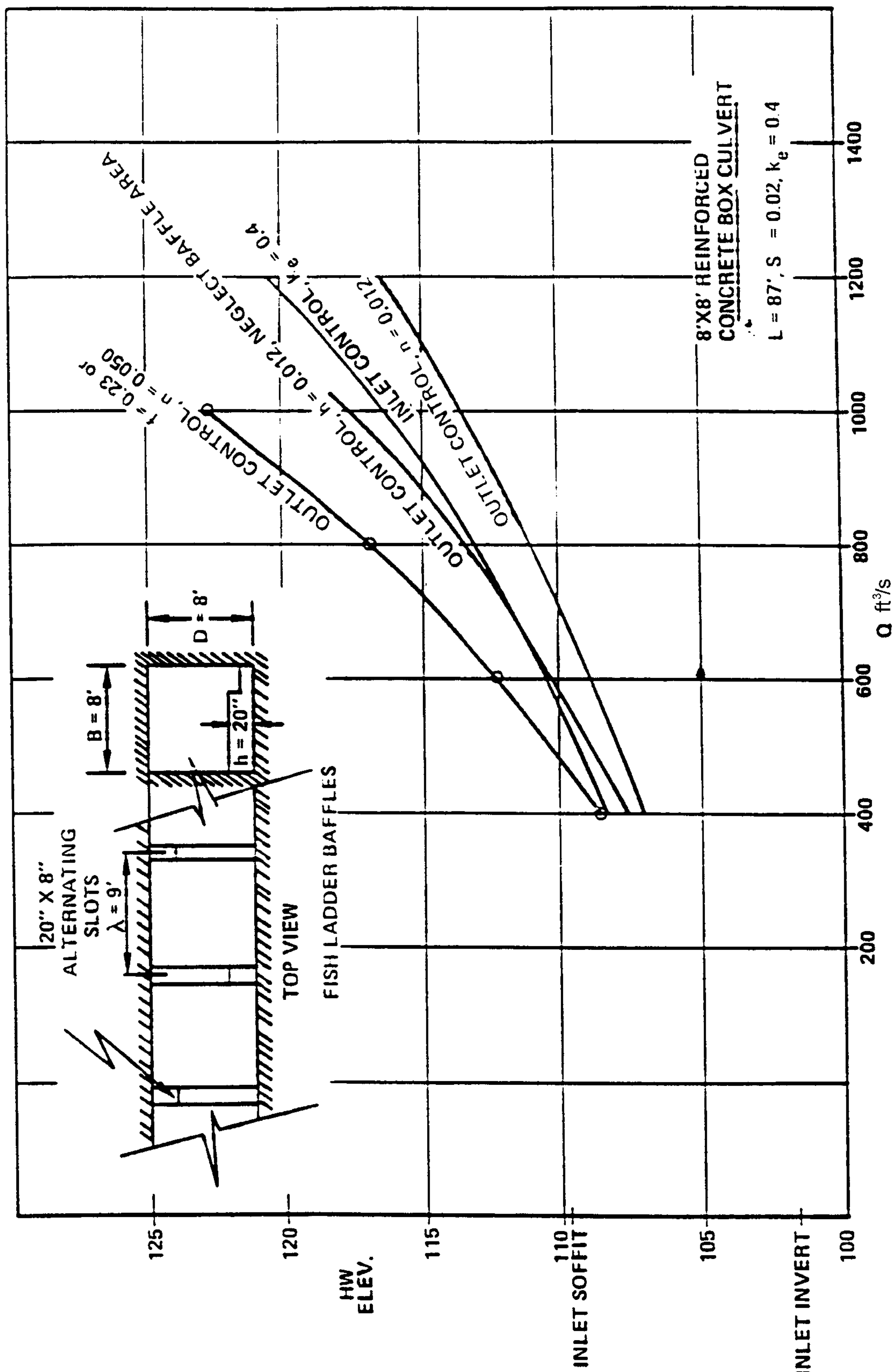

FIGURE 15.E-12 — Performance Curve

For comparison, rectangular baffles with the same spacing and height can be shown to have f = 0.23.

TABLE 15.E-16 — Discharge Versus Brink Depth

Q (ft^3/s)	h_o (ft)
353	6.3
706	7.8
1059	8.0
1412	8.0

**The calculated values of h_o based on d_c cannot exceed the top of the culvert barrel.*

Note, if the baffles with the rounded top edge had been used, with λ/D = 1.2 and h/D = 0.21, from Figure 15-16, f = 0.08.

For comparison, rectangular baffles with the same spacing and height can be shown to have f = 0.23.

15.C.7 EXAMPLE 7 — SLOT-ORIFICE CULVERT FISHWAY

Previous Examples illustrated the simple and complex methods for various fishway design and the use of a culvert performance curve. This Example focuses more on the slot-orifice aspects of a fishway design.

In this Example, two types of entrance slots will be considered:

- normal, and
- skewed.

This Example has three parts:

- narrated example,
- normal slot with computational form, and
- skewed slot with computational form.

<u>Narrated</u>

The first part of this Example will be narrated to illustrate the procedure. The remaining two parts will illustrate the use of a computational form:

Step 1 GEOMETRY. Assume a culvert to be 100 ft long with 6 slot orifices. The culvert slope is 2%. Further assume that both the culvert headwater and tailwater depths are found to be approximately 3 ft. From the procedure in Section 15.4.11, assume that the minimum required fishway width, B, was estimated as approximately 3 ft, and the contraction coefficient as m = 0.70.

Step 2 HEAD ON SLOT. ΔH = drop in elevation of water surface/number of slot orifices:

$$\Delta H = (0.02)(100)/6 = 0.333 \text{ ft}$$

Step 3 DISCHARGE. $H_i/T_i = 3.00 + 0.33/3.00 = 3.11$ and, from Figure 15-24 for a contraction ratio of m = 0.70:

$$Q/[BT_i^{3/2}(32.2)^{1/2}] = 0.106$$

Therefore:

$$Q = (0.106)(3)^{1.5}(32.2)^{0.5} = 9.38 \text{ ft}^3/\text{s}$$

Step 4 ORIFICE VELOCITY. Two methods may be used in computing the orifice velocity:

- with contraction coefficient, or
- omitting contraction coefficient.

With Contraction Coefficient — the approximate throat velocity $V_T = Q/[T_iB(1 - m)]$:

$$V_T = 9.38/(3)(3)(1 - 0.7) = 3.5 \text{ ft/s}$$

This velocity is compared to the swimming capability of the design fish. If the orifice velocity exceeds the burst or darting swimming velocity (see Table 15-5 and Table 15-6), the spacing of the orifices must be reduced (i.e., add additional orifices) until the actual and required velocities are compatible. Another alternative is to try a different contraction ratio, m, and redesign the fishway.

The mean flow velocity in the pool between the orifices must be less than the cruising speed of the design fish. The discharge, Q, divided by the mean cross sectional area, which is $[(B)(H_i + T_i)/2]$, will yield this approximate pool velocity.

Omitting Contraction Coefficient — Another common orifice formula is $V = 0.68[2g(\Delta H)]^{1/2}$, which may be used for computing the throat velocity of a vertical slot orifice. Using this formula for H = 0.33 ft, a throat velocity of 3.1 ft/s (as compared to 3.5 ft/s computed above) is obtained. The throat velocity is a function of the contraction ratio, m, and ΔH. For this reason, the procedure using Figure 15-20 provides a better estimate of throat velocity. Again, this velocity would be compared to the design fish's burst or darting swimming velocity to ensure that the two velocities are compatible.

Normal Slot

This Example illustrates the use of a computational form similar to that of Figure 15-21. Figure 15.C-13 reflects the fishway hydrology and site geometry. The computational table on Figure 15-21 is completed as narrated in Section 15.E.7.1.

$\phi = 90°$	m_s = N/A	Q = 10.2 ft^3/s
L = 50 ft	i = 6	Q/B (5.67) = 0.5996
S = 0.01 ft/ft	ΔL = 10 ft	S (ΔL) = 0.10 ft
B = 3.00 ft	T_i = 2.00 ft	
m = 0.70	H_i = 4.00 ft	

1	2	3	4	5	6	7	8
Orifice No. i	T_i (ft)	$T_i^{1.5}$ ($ft^{1.5}$)	$\frac{Q}{B(g)^{0.5}\,(T_i)^{1.5}}$	$\frac{H_i}{T_i}$	H_i (computed) (ft)	$T_{i+1} = H_i - S(\Delta L)$ (ft)	$V_i = Q/(B(1 - m)\,T_i)$ (ft/s)
						2.000	5.7
1	2.000	2.828	0.2120	1.382	2.764	2.664	4.3
2	2.664	4.348	0.1379	1.180	3.143	3.043	3.7
3	3.043	5.310	0.1129	1.123	3.417	3.317	3.4
4	3.317	6.042	0.0992	1.098	3.642	3.542	3.2
5	3.542	6.666	0.0899	1.080	3.325	3.725	3.0
6	3.725	7.190	0.0833	1.071	3.989		
				3.989 ≈ 4.00; therefore, Q = 10.2 ft^3/s OK			

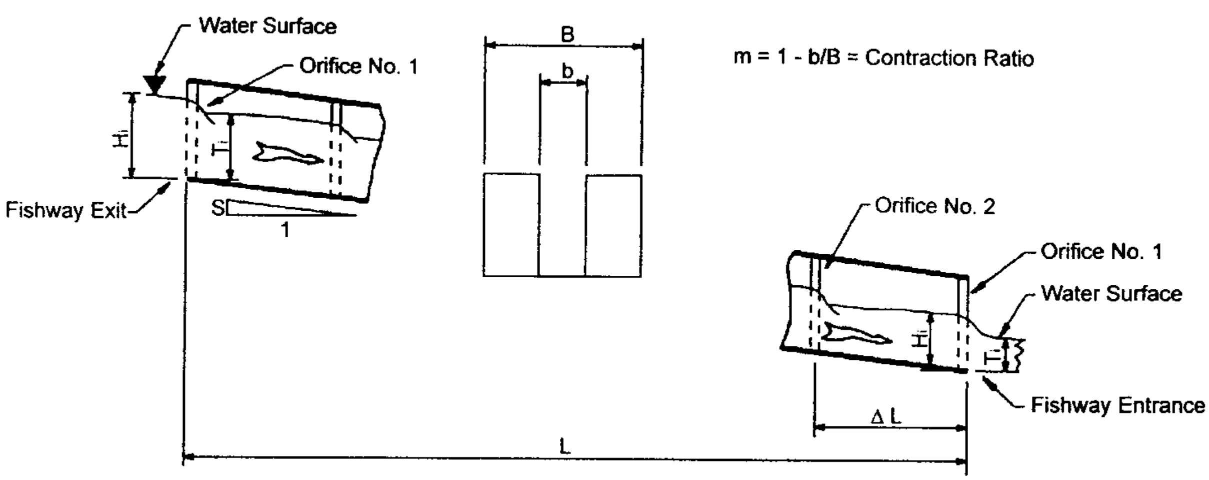

FIGURE 15.E-13 — Computation Sheet For Slot Orifice Fishway Design (Unskewed Entrance Slot)

Skewed Slot

This Example is similar to the Example in Section 15.E.7.2, only the entrance slot is skewed 45°. This Example is outlined on Figure 15.C-14.

ϕ = 45°	m_s = 0.75	Q = 14.3 ft^3/s
L = 60 ft	I = 7	Q/B (5.67) = 0.8407
S = 0.05 ft/ft	ΔL = 10 ft	S (ΔL) = 0.5 ft
B = 3.0 ft	T_i = 2.00 ft	
m = 0.70	H_i = 3.85 ft	

1	2	3	4	5	6	7	8
Orifice No. i	T_i (ft)	$T_i^{1.5}$ ($ft^{1.5}$)	$\frac{Q}{B(g)^{0.5}\ (T_i)^1}$	$\frac{H_i}{T_i}$	H_i (computed) (ft)	$T_{i+1} = H_i - S(\Delta L)$ (ft)	$V_i = Q/(B(1-m)\ T_i)$ (ft^3/s)
						2.000	7.9
1	2.000	2.828	0.2972	1.613	3.225	2.275	5.8
2	2.725	4.498	0.1869	1.312	3.575	3.075	5.2
3	3.075	5.392	0.1559	1.222	3.758	3.258	4.9
4	3.258	5.881	0.1429	1.190	3.877	3.377	4.7
5	3.377	6.206	0.1355	1.170	3.958	3.458	4.6
6	3.458	6.430	0.1307	1.162	4.014	3.514	
7	3.514	6.587	0.1276	1.090	3.830		
				3.83 ≈ 3.85; therefore, Q = 14.3 ft^3/s OK			

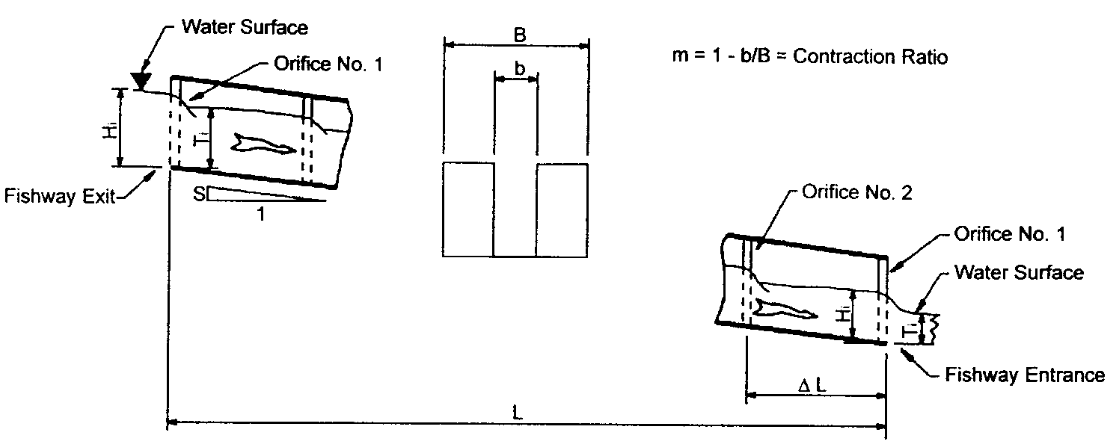

FIGURE 15.E-14 — Computation Sheet For Slot Orifice Fishway Design (Skewed Entrance Slot)